



**First and Second Interim Reports
of the Bridge Sub-Committee, .
1925, on "Impact" & "Revision
of the Bridge Rules,"
with connected papers**

and

**Proposed Bridge Rules for 5'-6"
Gauge Railways**

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L. E. HOPKINS,
Director, Civil Engineering, Railway Board.

DELHI;
December 1925. }



1

TERMS OF REFERENCE TO THE BRIDGE SUB-COMMITTEE.
(WITH NOTES.)

The object of the inquiry is to ascertain as rapidly as possible in what way and to what extent the cost of Steel Bridges for the passage of Trains can be reduced.

The Indian Railway Bridge Committee, convened in 1917 for the purpose of revising the Bridge Rules, carried out a number of valuable investigations, with particular reference to the subject of vibration of beams. The Railway Board felt that, although this had an important bearing on the Bridge Rules, it did not appear that the Committee were following a line of investigation which was likely to lead to definite results within a reasonable period. They, therefore, closed the work of the Committee in order that they might take stock of the position and decide what further action was necessary.

In this connection the report of Mr. Lloyd-Jones on his tour in America, and the account he gives of his interviews with American Engineers is extremely interesting. The Engineers consulted may be divided into three classes: There are a large number of practical and busy men who have, no doubt, designed many bridges with the Pencoyd Formula, or rules or formulæ of their own, and find that their bridges are quite safe, but, like many Engineers, they have not deeply studied the subject, and they are willing to admit, when asked, that it is possible further investigation may be able to teach them something about their work, and they are willing to admit the necessity of further experimental research especially when asked by a visitor deputed by a foreign government. Their opinion may be read in the light of the above remarks.

Then there are the Professors headed by Professor Turneaure, who probably have little practical experience of design of railway girders but who, like all scientific men, are attracted by the idea of finding a mathematical solution of such problems as the vibration of beams. It may be noted, however, that Professor Turneaure naturally takes up a modest view of his previous work when he is consulted by a distinguished foreign Engineer and admits that his own investigations are merely pioneer work.

Lastly, there are a number of practical men such as Mr. Gustav Lindenthal, who is described as the leading Bridge Engineer of the world, Mr. Reichmann, and Mr. Richard Khuen of the American Bridge Coy., and Dr. Waddell and others and most of these gentlemen discourage the idea of further experimental research. Mr. Lindenthal, in fact, is the only one of all those consulted in America who offers a strong opinion on the subject and his remarks should be carefully studied.

Turning again to the first paragraph of the terms of reference, it would seem to be necessary for the Committee to commence their investigations by making a thorough analysis of the weights of girders designed in accordance with the various rules which have been in use in different parts of the world besides this country. They should, in fact, design girders of 20, 40, 60, 100, 150, 200 and 350 ft. spans for B Loading, B + 65 % Loading and 2 B Loading, using the Rules of 1892, of 1903 and of 1923, also the Canadian Bridge Rules, Lindenthal's American Bridge Rules and Ministry of Transport Rules, and they should also see the effect of making no allowance for Impact at all. These Rules all differ vitally in various ways, not only is the Impact Formula varied widely, but the stresses in some cases include the effect of wind, in some cases do not include it, and in some cases allow for secondary stresses and in others they do not, and so on.

In the 1892 rules rivet holes were deducted in calculating the area of compression members. All these designs should be tabulated and analysed and it will possibly be found that there is little difference between one Impact Formula and another, and that it is not worth while quarrelling about the relative advantages of the Pencoyd Formula and the British Engineering Standards Formula, in fact in one case the proportion of stress due to the live load appears to be about 26 per cent. of the elastic limit, in the other about 28 per cent.* In any case after compiling all these data the Committee will be saturated with the point of view of Mr. Lindenthal, tempered by sympathy with the aims of Professor Turneure. In short, Mr. Lindenthal's view is probably influenced by the fact that very large numbers of bridges all over the world have been designed with all sorts of Impact Formulæ, and that so far none of them has failed from Impact, and there is, therefore, available a large mass of actual evidence, the meaning of which would be clear after the Committee have obtained a practical experience of all the different methods and results of designing.

Finally the Committee would read what the Government of India wrote in 1892 when introducing the Bridge Rules of that date.

"The coefficient, if accurately determined, would not only differ for each member of the same bridge, but would again differ for corresponding members of bridges of different sizes or different designs. It would probably differ further to some extent with changes of temperature, and would certainly again differ for different designs of engine and different speeds of train."

"Hence it is generally admitted that to compile a specification, which shall accurately and consistently apportion the working stress to every case which may occur (even under ordinary conditions), is quite out of question. The rule as drafted must therefore be an attempt to arrive at a compromise, which shall secure an approximate accuracy sufficient for practical purposes, without tendering necessary unduly complicated calculations."

"There must be a compromise and the question for decision is what compromise shall be adopted."

The general proposition is now the same as in 1892 except that a different view may be taken of the nature of the compromise and more data are available to enable a judgment to be made.

The principal reason for supposing that the present impact coefficients are too high is the fact that girders are designed for the static load produced by the heaviest engine, and then that load is increased by an impact coefficient produced by a different engine which produces higher impact but with a smaller static load effect. In addition to this the problem is to decide how many of the experimental results may be ignored which may be left to the factor of safety to take care of. The Committee would have to decide what is the reduction in weight and cost of steel bridges which may be looked for, and it would seem that unless a reduction of 15 per cent. can be effected it is not worth while discarding the existing Pencoyd Formula. In order to get a 15 per cent. reduction some such formula as $\frac{70}{40 + L}$ or $20 + \frac{1200}{10 + L}$ would be required and if these are plotted on a graph of all the available experimental results eliminating those made by engines giving high impact and low static effect and exclude more than a reasonable percentage, it is for the Committee to decide whether it is worth while pursuing the subject.

2. What general formula for impact shall be adopted in future?

It will be necessary to marshall the known facts *so far as they are relevant*. It has been known for many years that the principal cause of impact for all spans is the effect of the overbalance weights in the locomotive drivers. In the case of short spans the variation of load due to this

* In the case of a 100 ft. Truss.

cause can be determined with considerable accuracy. In the case of long spans where the effect is due to the vibration set up by the periodic force, it is impossible to arrive at an exact solution of the maximum effect of these forces or at any rate within a reasonable time. It has been suggested that the problem might be further investigated by fixing a locomotive in the centre of a girder and recording the effect of the vibration set up while the drivers revolve. It does not seem that this would reproduce the effect of the same engine moving over the bridge, and therefore, the result of the experiment would not appear to lead to any practical solution of the problem. Fortunately it is not necessary to arrive at an exact solution of the problem, or find out how many times in ten thousand such maximum effect is likely to occur. All it is required to know is the probable ordinary maximum effect, and occasional effects may be left to the factor of safety. Bridges cannot be designed to cover maximum effects of impact, and there must always be a factor of safety to cover natural and personal errors. Nor, if it is known exactly what is the impact effect of each locomotive in use, would it be possible to issue regulations for the drivers to observe certain speeds in passing each type of girder, of each span length. Further knowledge available is that the effect of impact is proportional to the mass of the girder, and to its depth, and breadth, and length, and it is for decision whether the formula for impact will retain its simplicity while including factors for the locomotive drivers, the mass, the depth and the breadth and the span. Hitherto formulæ have usually had one variable either the length or the mass ($\frac{L}{L + D}$). Mr. Lloyd

Jones has proposed a formula, which will include most of these variables, which he lumps together as engine factor and bridge factor; and Professor

Inglis has actually produced such a formula $\frac{2 P l^3}{\pi^4 E. I.} (2 N n^2)$ which he wishes to have proved experimentally. It is thought that a formula of this nature cannot become a practical proposition. It is too clumsy; it will take many experiments over a long period with a varied assortment of bridges and engines to establish; it would logically require a driver to observe variable speed regulations over the girders on his section; and it would still omit to take into account several factors in the problem such as the differences in the effect of impact on members in trusses according to their position,—the effect of oscillation in a horizontal plane, and the various causes of impact which have nothing to do with vibration of beams. A formula of this nature would be interesting and useful in analysing old girders, but it would require lengthy investigation to establish successfully, and the whole idea would tend to confuse the aims of the Committee, which should be to apply what knowledge is available in removing the causes and modifying the effects of impact, rather than in providing for girder designs which are unsound and uneconomical, for the use of engines which are defective in design and likely to become obsolete in the near future. The value of the formula is in enabling the experimental results of the past to be analysed, and those results due to engines which have a high impact effect on a low static effect to be set aside.

The anticipation in Rule 28 (a) of the Bridge rules of 1923 that a formula will be evolved independent of the span and depending solely on the engine axle-load is difficult to understand. The span is the most important of all the variables, and if the proposal means anything, it appears to mean that having once fixed the standard of loading the percentage for impact is also fixed.

Assuming that the Committee arrive at the conclusion that the span is the only necessary variable (not the loaded-length) *it will be necessary before fixing the constants in the formula to decide what restrictions or rules should be laid down about the remaining variables.* Before dealing with these matters there are two other points to be considered about the formula. Does the maximum impact effect occur in any member at the same time as the maximum stress due to the live-load, and does the effect of impact vary with the position of the member. If a factor is used based on the span, or on the maximum impact effect on the span or the

member, it has the effect of giving a high impact coefficient to members which may not be highly stressed at the moment of maximum impact. If it is based on the loaded length, these members are given a still higher impact coefficient. These questions will need consideration in fixing the form of the impact formula, and in their consideration it is for decision whether experiment is necessary to decide how impact effects are distributed over the different members of a truss. The Committee may next turn attention to the regulation (paras. 3 to 7) of those factors which for the sake of simplicity are to be left out of the Impact formula, but which must be decided before the constants in the formula can be fixed. The conclusion of Professor Inglis' pamphlet should be read. Here he gives his opinion that impact formulæ based on live-load are no use, that formulæ based on engine axle-load are not much better and says that one should "select the worst offenders" among locomotives on a railway and investigate them to arrive at one's own impact coefficient. It is thought that these difficulties will disappear, and a practical formula based on the span will result, if the procedure here outlined is followed.

Lay down rules which will result in the most favourable treatment of those variables which are capable of treatment, *e.g.*, the depth, breadth, mass, and periodic force, (the span cannot be got rid of by rules). Secondly, the "worst offenders" in locomotives may be put on the scrapheap and no more built. Thirdly, the perfect balancing of locomotives may be paid for. Fourthly, remove the mistake by which members of girders have been designed for static loads produced by the heaviest engines, and these loads have then been increased by a percentage of impact produced by an entirely different engine, giving less static load, but higher dynamic augment, and moreover producing this maximum impact in some members at a time, when they are not fully loaded statically.

MASS.

3. Do you recommend a rapidly increasing impact factor for short spans?

The effect of impact of two bodies is inversely proportional to their masses and it is generally considered that facts support theory in this case and that effect of impact for short spans is proportionately much greater than on long spans.

MASS.

4. Do you recommend the use of high tension steel for the construction of medium and long span bridges?

In the case of medium and long span bridges, the use of high tension steel might reduce cost by reducing the dead-load, and in the case of medium span bridges the impact effect due to amplitude of vibration may be reduced by reduction of the mass, and this might be worth considering.

LENGTH AND DEPTH.

5. Do you recommend the adoption of a minimum depth for plate girders of $\frac{1}{10}$ th span and trusses $\frac{1}{6}$ th span or alternatively that girders be made strong enough to deflect no more than if they were designed with these depths?

Theory states that the amplitude of vibration of a beam varies as the cube of the span and inversely as the moment of inertia. It follows that the effect of impact may be reduced by making the depth as great as practicable. With the heavy loading now customary of standard 1'65 or 2-B a depth of $\frac{1}{6}$ for trusses is now practicable. The economic depths may be taken as $\frac{1}{6}$ for plate girders and for trusses and these depths should be used where possible.

BREADTH.

6. Do you recommend that the distance apart of trusses and girders be $\frac{1}{10}$ of the span with a minimum of 8 ft.?

The object of this rule would be to ensure a greater resistance to oscillation for reasons similar to those referred to in 5.

PERIODIC BLOW OF THE OVERBALANCE.

7. The Dynamic augment on all the coupled drivers shall not exceed 50 per cent. of the static load on the main driving wheel.

But until details of balancing of existing engines are available the proposal is provisional.

In order to ensure the correct balancing of 2 cylinder engines which are still being built, a rule of this nature is required. The object of this rule is to reduce the value of the periodic force as far as possible and to make it a uniform percentage of the static load. This will make it possible to use the same impact coefficient for all locomotives. It may be argued that the rule will not have this effect; but it will give the nearest approach to that result which is possible.

If passenger engines cannot be built with reciprocating parts of high tension steel light enough to meet these requirements, or if they sway too much, if less than half the reciprocating parts are balanced, then 3 or 4 cylinder engines must be paid for, and old engines specially penalised or restricted till scrapped. At present there is no information in this country of the weights of reciprocating parts of locomotives, nor how much has been balanced. The Consulting Engineers have been asked to obtain this information for the use of the Committee.

8. Do you recommend that short spans ballasted or provided with heavy timber floors should be allowed a reduction of impact factor?

This question may be examined on the same lines as 3. But it is probably of small importance as the design of small spans is usually more affected by practical consideration of selection of a suitable standard size of joist than by minute calculations of impact.

9. In designing girders to carry Locomotives free from reciprocating parts is it necessary to provide for impact and is it possible to design 3 and 4 cylinder Locomotives in such a way as to place them in this class?

Professor Dalby writes "the reciprocating masses in a 4-crank locomotive may be arranged to balance without the use of balance weights. Under these circumstances there will be no variation of rail-load and no horizontal swaying couple; the engine will be perfectly balanced neglecting the error due to obliquity of connecting rod." He also states that in the Deutschland a ship 662 feet long with engines of 37,000 H. P., perfectly balanced by Mr. Schlick's method the maximum amplitude of vibration was only $\frac{3}{16}$ inch. Two cylinder motor car engines were discarded very early in the history of motoring because of the balancing difficulty, but two cylinder locomotives have survived 100 years, and the Committee here will have an opportunity of turning to practical use the knowledge, which has long been available, that vibration is due to badly balanced engines. On long spans the Committee will probably be able to decide that locomotives without reciprocating parts may be treated in the same way as dead-load, but whether short spans below 30 ft. may be treated in the same way is for consideration. If the Committee can recommend a rule to carry out the ideas in this reference, they will have effected a great reform. It would seem that this is the most important reference with which the Committee have to deal.

10. Do you recommend any alteration in the permissible stresses in steel laid down in the B. E. S. A. rules?

The rules as modified by the Bridge Rules now permit apparently,

(8 tons)	18,000 lbs.	say, per sq. in. in tension for D. L. and L. L. and imp. and wind, centrifugal and braking forces and temperature.
plus	7,000	secondary stresses. This appears to be the intention of paras. 12 and 13 of B. S. S. No. 153(3).
plus	6,250	for old bridges which it is desired to retain for financial reasons temporarily. This is the effect of footnote page 18 of Rules of 1923 and last paragraph, page 6 of Vol. III of Progs. I. R. B. C.
25 per cent.		
Total	31,250	
plus	2,250 (say)	subject to restrictions of 10 miles per hour; or it would be more correct to say that, if the stress as calculated above amounts to 33,500, it may be reduced to 31,250 by a speed restriction of 10 miles per hour.
$\frac{1}{2}$ impact.		
Total	33,500	

The bearing and shearing stresses should be considered, and if necessary recommendation for alteration may be made.

11. Do you recommend any other alteration in the rules?

A revised draft of the whole of the rules may be given if considered desirable—

- (a) The question of retaining the B. S. loading may be considered, bearing in mind the probable introduction of increased running dimensions of Broad Gauge stock, and the need for considering the convenience of the British Bridge and Steel Industry, which for many years is likely to remain our principal source of supply of steel work.
- (b) The definition of secondary stresses might be altered to an explanation in simple language, *i.e.*, stresses due to distortion of a truss under deflection owing to rigidity of joints, and those due to eccentricity at the joints.
- (c) It may be desirable that Rule 5 (c) should have a standard form showing an abstract of the calculations required for each member.
- (d) *Rule 14.*—The definition requires entirely redrafting. The effect of impact is not only vertical, it includes horizontal oscillations.
- (e) The table of uniform loads for standard B may be supplemented by loads for standard B+65 % and for a 2B standard.
- (f) Chapter II 3 (c) these tests should be made with the heaviest passenger engine on the line.

L. E. HOPKINS,

Director, Civil Engineering.

November 24th, 1924.

II

REMARKS BY DIRECTOR OF CIVIL ENGINEERING AT THE OPENING MEETING OF THE BRIDGE SUB-COMMITTEE.

You have read the proposed terms of reference and I hand you now a *precis* of the criticism received on them and I will take the opportunity to explain briefly how I think you should tackle the Impact Problem and what the final terms of reference are.

In the first place, the Committee should always keep in mind that they are a body of practical men dealing with a practical proposition, the cost of girder work. By practical man I mean, for instance, that they will consider real trains at working speed. For instance, a full B+65 % train could hardly be run at over 30 m.p.h. on spans of 150 ft. and over. This is not a scientific investigation and nothing more is needed to solve the problem than pens, ink and paper and simple arithmetic and clear thinking.

2. The next point is that further experiments with new and highly developed stress recorders will not solve the difficulties. There are many hundreds of experiments on record and the solution of the problem must be contained in them quite plainly if we can only see it. If the answer cannot be extracted from the existing experiments, I fear you never will find the answer. I do not say that some experiments may not be necessary later on to confirm and revise, but I am certain that the solution on general or approximate lines can be obtained from the existing information. Until the existing experiments have been correctly analysed, sorted, tabulated and results got from them nothing can be gained by making more experiments. The collection of all the data available of the Indian experiments is far from complete and in sorting, tabulating and analysing there is much to be done. The American experiments are fairly clear and only need editing and arranging and some alteration.

3. The next point is to decide how to analyse the experiments so as to bring out the meaning of apparent disagreements. It is in these disagreements and apparent discrepancies that the answer to the problem lies. In the notes to terms of reference I have tried to indicate what are the variables and in finding the value to be given to each variable is the solution of the problem. The important variables are:

- (1) The Hammer Blow. I think this term is preferable to Dynamic Augment. In general it is advisable to employ common terms wherever possible.

The Hammer Blow should be ascertained at the maximum working speed with full load, and at the critical speed. The combined Hammer Blow of all the coupled wheels should be taken, it is only on the very short spans, up to 12 feet that the Hammer Blow per wheel is required.

Experiments with goods engines at 50 m.p.h. where maximum working speed is 35 m.p.h. are unnecessary. Maximum working speed of passenger engines is 70 m.p.h.

- (2) The mass or weight of the bridge and its relation to some standard which for convenience may be Standard B.
- (3) The weight of the test train and its relation to Standard B. The test trains will have to be reduced to equivalent loading for this. Particulars are given in most cases.
- (4) The proportion of the Hammer Blow to the actual total load of test.
- (5) The proportion the Hammer Blow bears to the total load for which the bridge is designed.

cf. Prof. Inglis who says—

$$\text{added deflection} = \frac{2}{\pi^4} \frac{Pl^3}{EI} 2 N n_0^2$$

$$\text{but } n_0^2 \propto \frac{1}{\text{defln.}} \text{ i.e., } \propto \frac{EI}{Wl^3}$$

$$\text{Therefore added defln.} \propto \frac{P}{W}$$

$$\text{i.e., } \frac{\text{Hammer Blow}}{\text{Total Load}}$$

4. Having got all these particulars worked out for all the experiments and conveniently tabulated, results of all the experiments should be plotted, and each result examined and its value estimated in relation to the remainder.

For instance, the experiments on the 440' span were made with a passenger engine with 7 ft. drivers. At the critical speed of 20 m.p.h. the Hammer blow was $\frac{3}{4}$ ton while the total load was 2,000 tons, the ratio of Hammer blow to the total load .038 per cent. On the other hand, the Kotri bridge was tested with a 2-8-0 goods engine without any train load behind it which could not run fast enough for the critical speed and gave a Hammer blow of nearly 5 tons at full speed and the ratio of Hammer blow to total load was .7 per cent.

Clearly the effect of the Hammer blow on the 440' span could hardly have been more than the effect of the jar at a bad joint, while the high ratio in the Kotri case indicates one of the causes of high impact coefficient if not the only cause of importance. The last Bridge Committee saw this point and mentioned it on top of page 6 of Vol. 5, but they did not give it sufficient weight. It is, I think, of vital importance.

5. On page 2 of the terms of reference, lines 3 to 10, I have alluded to the principal cause of the inaccuracy of present impact formulæ and I will explain this further. It is important to realise that impact coefficient is percentage of live-load. In other words, if the live-load is a small proportion of the loading for which the bridge is designed, it does not matter if the impact coefficient is high. At the same time when the live-load is much below the standard of the Bridge the critical speed is high and consequently impact coefficient is high and for this reason experiments with test trains below standard are usually misleading. The absurdity of experiments with single engines over bridges over 30 ft. spans and especially on long spans can be seen if the test train be still further reduced. Instead of using a 2-8-0 engine in the Kotri experiments suppose the tender to be cut off and the boiler removed and the chassis fitted with a compressed air cylinder to give enough power to run the chassis over the bridge and enough weight to keep it on the track.

The crawl stresses would then be scarcely observable and the critical speed would be 50 miles or more an hour and the Hammer Blow would be about 9 tons and impact factor might be anything up to 5 times instead of 20 per cent. especially on web members. (The fully loaded bridge critical speed is 15 m.p.h. for this engine Hammer Blow $\frac{3}{4}$ ton, load 1,200 tons).

At the same time the bridge would not be overstressed even with a 5 impact factor. It would be just as absurd to adopt the impact factor obtained in this way as it is to adopt the results of the test with an engine without loading the bridge fully. It follows from this that impact tests must be made with a train equal to the full calculated live loading of a bridge.

It also, I think, follows that the heavier the loading the less proportionately the impact. Clearly the effect of overbalance on a Standard 2-B bridge will be less than on a Standard B, because the hammer blow will be less in proportion to the total load.

It further follows from these considerations that the proposals of the last Bridge Committee to evolve an impact coefficient based on the engine axle-load and practically independent of both the train load and the total load were wrong in principle.

6. As regards the effect of depth ratio on impact the experiments cover a wide range of girder with depth ratios from $\frac{1}{4}$ to $\frac{1}{12}$ and there should be sufficient data therefore to arrive at a conclusion on the effect of depth.

As regards the general importance of the impact factor as explained in my previous notes, it will be necessary for you to design a number of girders and tabulate the results till you get a clear idea of the value of impact relative to the other stresses.

7. As to Hammer blow you will require to collect particulars of all locomotives possible and work out the Hammer blow of each at working speed. You may be able to come to a conclusion as to the limits which balancing should not exceed.

It is probable that for passenger engines the Hammer blow at working speed 70 m.p.h. on all the coupled wheels together, will have to be permitted to equal the load on the main driving wheel, but for goods engines the Hammer blow on all the coupled wheels need not exceed 60 per cent. of the load on the main driving wheel or perhaps less.

In the case of passenger engines half the reciprocating parts and $\frac{1}{4}$ connecting rod have to be balanced, but in goods engines it would be perhaps sufficient to balance even less than this.

There are other considerations about balancing such as ratio of stroke to diameter which seem worth investigating, also ratio reciprocating parts to tractive force, it would seem that this ratio gets less as tractive force increases.

The consideration of impact for 4-cylinder engines may be left till the impact for unbalanced engines has been settled.

8. I have given you a line of investigation leading up to a definite aim, if you know of a better one I shall be glad to hear of it, but so far I have not heard of any other clearly thought out proposals for elucidation of the impact experiments.

It is essential that you should start work with a definite aim in view and not allow yourselves to be put off it by side issues.

I will recapitulate the programme :—

- (1) You have to make proposals for reducing weight of girders for a given loading by revising the impact coefficient.
- (2) Therefore you want to find out how much of the weight of girders is now due to the impact coefficient under various rules and how much is due to other allowances.
- (3) Next you have to make proposals for an impact factor and for this purpose you have to decide what kind of engines and bridges you are to cater for as obviously there are great differences and you want to know that the worst engines and bridges for impact are ruled out.
- (4) This brings you to study the different variables which are :
 - (a) live load, dead load and critical speed;
 - (b) Hammer blow and its ratio to load;

do you agree that these are the controlling factors? if so, you then work out the particulars for all the different experiments available.

- (5) You then examine each experiment in the light of the particulars and plot as many as possible on a diagram. Lay off a curve for impact coefficient to a formula of, say, $\frac{70}{40+L}$ or as low as you dare and then examine every experiment which exceeds this curve and see if it can be explained, as the result of improper loading. That is to say an experiment with a light engine giving a critical speed of 60 m.p.h. and high impact percentage when a full load would have had a critical speed of 15 m.p.h. and low percentage impact; or an experiment with a Standard B train on a B+65 per cent. Bridge; or an engine which was so badly balanced that the rules you have laid down could not allow it on the line in future.
- (6) If this method is successful in permitting you to recommend a formula much lower than the Pencoyd say $\frac{70}{40+L}$ for engines with a definite balancing rule then your task is complete and you can start on the question of secondary stresses, rivet stresses in steel, B. E. S. A. rules, etc., for which new terms of reference will be given you. But till then it would be better to avoid all such issues.

9. It is clear then that the Committee will have no occasion for much immediate discussion. It will take them some time to tabulate and prepare all the experiments and calculate a series of girders. They will divide the work up amongst themselves and when complete they will have the results brought together and printed.

They will then spend some time studying them individually and discussing them with others: may think it necessary to send the results to other Engineers and go on tour and discuss with other Bridge Engineers and finally meet together and come to a conclusion.

There is *prima facie* evidence that wide spacing of stringers with heavy bridge timbers in addition to Hammer blow have a considerable influence on impact, but I have no intention of trying to forecast your conclusion, but I feel sure you will have no difficulty in finding the answer to the impact question. When that is done you can consider the question of secondary stresses and other matters.

The Railway Board will be glad if you will send in your report not later than July 15, 1925.

The criticisms received up to date on the main features of the terms of reference are noted on in the following pages. The full criticisms will be printed with my notes (printed separately).

L. E. HOPKINS.

DELHI;

The 26th February 1925.

III

BRIDGE SUB-COMMITTEE, 1925.SUMMARY OF COMMITTEE'S ANSWERS TO THE TERMS OF REFERENCE.

Nos. 1 and 2.—*What general formula for impact shall be adopted in future?*

The Committee have first produced a basic formula for impact applicable to existing bridges which contains all the variables which are believed to be important in affecting impact in practice. This formula is

$$I\% = \frac{1050 P_1}{(w+p) c d}$$

for single engines and must be multiplied by 2 for double engines.

They have then produced a general covering formula for impact on capacity loads for use in design, viz. :—

$$i = \frac{65}{45 + L}$$

where L=loaded length in feet. This formula is obtained from the basic formula.

The investigations have chiefly been carried out in reference to Broad Gauge bridges. (See Appendix F on its application to Metre Gauge bridges).

MASS.

No. 3.—*Do you recommend a rapidly increasing impact factor for short spans?*

The formula provides for a rapidly increasing impact on short spans.

No. 4.—*Do you recommend the use of high tension steel for the construction of medium and long span bridges?*

(a) The question of adopting high tensile steel for medium and long span bridges may have an economic basis, but it is doubtful whether its use is justified for spans not exceeding 350 feet. At present high tensile steel is considerably more costly than ordinary commercial mild steel. It is understood that a high tensile steel has been produced and adopted in Germany quite recently and it would be advisable that its development should be watched.

(b) As regards the effect of mass on the impact that may be expected, it is shewn in the basic formula that increase of mass reduces the impact and *vice versa*.

LENGTH AND DEPTH.

No. 5.—Do you recommend the adoption of a minimum depth for plate girders of $\frac{1}{10}$ th span and trusses $\frac{1}{6}$ th span or alternatively that girders be made strong enough to deflect no more than if they were designed with these depths?

The effect of increasing depth of a girder of given span is to reduce its deflection and hence the critical speed and the corresponding impact are raised. This appears in the basic formula and is explained in the report. Hence from the impact point of view increased depth is a disadvantage.

A rigid rule does not appear to be necessary.

BREADTH.

No. 6.—Do you recommend that the distance apart of trusses and girders be $\frac{1}{15}$ th of the span with a minimum of 8 feet?

A rigid rule to this effect does not appear to be necessary and may be uneconomical.

In the case of reconstruction of bridges where superstructures have to be fitted on to existing piers it will frequently be difficult to adhere to the ratio of one-fifteenth. In certain cases where the distance between girders has to be restricted, lateral rigidity can be specially provided for by a suitable bracing system.

Most specifications give the ratio as one-twentieth.

Regarding the particular case of stringers, there was some evidence in the American tests (*vide* plate II-g in Bulletin No. 125 of A. R. E. A.) of damping of secondary vibrations from the employment of wide spacing, but it is not clear that the primary vibrations are reduced and that the benefit, if any, is sufficient to warrant the additional cost involved and the maintenance.

A further consideration is the risk of placing reliance on the strength and durability of the bridge timbers.

PERIODIC BLOW OF THE OVER-BALANCE.

No. 7.—The Dynamic Augment on all the coupled drivers shall not exceed 50 per cent. of the static load on the main driving wheel?

The Committee have shewn that the correct criterion for judging the "out-of-balance" of an engine is the value of $\frac{P_1}{c}$ and that for standard II this should not exceed .041 and for Standard III .0542. (See Appendix C. Note on application of impact formula to Standard III loads).

No. 8.—Do you recommend that short spans ballasted or provided with heavy timber floors should be allowed a reduction of impact factor?

There is not sufficient evidence to pass a definite opinion as to reduction of impact in the case of short spans.

No. 9.—*In designing girders to carry locomotives free from reciprocating parts is it necessary to provide for impact and is it possible to design 3 and 4 cylinder locomotives in such a way as to place them in this class?*

No. 10.—*Do you recommend any alteration in the permissible stresses in steel laid down in the B. E. S. A. rules?*

This is dealt with in paragraph 26 of the report.

The question of working stresses is one to be determined in relation to the quality of steel specified. It is a broad issue and the Committee do not see their way to make a definite recommendation excepting in the particular case of rivet bearing stresses. The Committee find that, for old girders left in the road without speed restriction or strengthened and used in other place, there is evidence to shew that the permissible bearing stress in the rivets connecting the flange angles to the web of plate girders may be safely taken as 18·0 tons per square inch if calculated by the B. E. S. A. formula corrected for the local effect of the wheel load on the flange. The corresponding permissible stress for wrought iron is 15·0 tons per square inch. (See Appendix G showing particulars of calculated bearing stresses in girders on the Madras and Southern Mahratta Railway which have been in use for many years and shew no signs of loose rivets).

No. 11.—*Do you recommend any other alteration in the rules?*

The question of Bridge Rules is a comprehensive one and is under consideration.

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IV

To

THE DIRECTOR OF CIVIL ENGINEERING,
RAILWAY BOARD.

Simla, the 6th June, 1925.

DEAR SIR,

The Bridge Sub-Committee submit herewith their Interim Report on the work done to date. The only subject which has been tackled is No. 1 Impact.

A formula for calculating the Impact to be expected from any locomotive on any bridge has been evolved. This has been tested by applying it to the available records of tests and it has been found that it gives excellent agreement for the Indian Broad Gauge Tests and good, but not quite such good, agreement in the American Tests. It explains away most of the hitherto puzzling discrepancies.

This formula has then been applied to evolve a covering formula for design, taking the locomotive appropriate to the Standard of Design adopted: and it results in a formula of $\frac{65}{45+L}$ for coupled Engines of Standard II. It is pointed out that this represents a reduction on the Pencoyd Formula. This reduction increases gradually to 50 per cent. (or more) on spans beyond 150 feet.

The Committee believe that the same formula can be used on any gauge, but there are no reliable tests by which they can verify their opinion.

There are several other points, for example, double track bridges, which the Committee have not been able to consider because they were requested to put in an Interim Report in time for inclusion in the Agenda of the Meeting of the Standing Committee of Chief Engineers.

Yours faithfully,

H. N. COLAM,

F. HARRINGTON,

L. H. SWAIN,

} *Members,*
Bridge Sub-Committee.

MUZAFFAR HUSSAIN,

Secretary, Bridge Sub-Committee.

V

BRIDGE SUB-COMMITTEE, 1925.

FIRST INTERIM REPORT—SUBJECT I—IMPACT.

1. *Definition.*—The Impact Allowance on Bridges is herein taken to cover only the effect on the bridge of the load moving over it at speed, over and above the effect of the same load crossing the bridge at a crawl.

2. *Measurement.*—It can be measured either by the additional deflection of the bridge as a whole, or by the additional stress caused in any member. But instruments attached to a member fail to register the average stress in that member unless certain precautions are taken to avoid recording local effects, secondary stresses, etc. Unfortunately in most of the thousands of tests the results of which have been published, these necessary precautions have been omitted. Consequently very few of the tests with stress recorders can be used with any confidence. On the other hand records of deflection of the span are not liable to these errors. They are remarkably consistent and may be taken fairly to represent the average impact effect in all the members of the span. For this reason the present investigation has been confined practically to the determining of the correct impact allowance from deflection tests only.

3. To meet the views of those who hold that stress tests must be considered, a number of selected cases have been examined, where it was considered that the precautions necessary for arriving at the average stress in the member had been taken. The conclusion arrived at is that impact determined from stress tests is generally higher than that determined from deflection tests by about 10 per cent. See Appendix (B). But as the calculated impacts (calculated in the manner explained later) exceed the observed impacts by more than 10 per cent. on the average, it is considered that they may be left to take care of this difference.

4. *Causes of Impact.*—The causes of impact may be classified as follows:—

(i) A periodic vertical force on the bridge caused by the engine and synchronous with the revolutions of the driving axles. It seems to be fairly represented by the resultant of the centrifugal forces due to the counterweights which are used to balance the reciprocating parts. The resultant centrifugal force is called for convenience hereafter the "hammer-blow." This is not to be confused with the individual hammer-blows delivered on the rail by individual counterweights. It is the resultant sum of them all.

(ii) Rolling load effect. This is the effect of vehicles considered simply as rolling loads which in themselves on smooth level track produce a constant force at the rails. When crossing a bridge on which the track has negative camber, the resulting centrifugal force may cause additional deflection. Also if the camber on the rail is excessive additional deflection may be caused.

(iii) Irregularities of track, oval wheels, flats on wheels, etc.

5. The effects produced on the bridge by these various causes on short spans and bridge floors are very different from the effects on the main girders of long spans, and for this reason it is proper to consider "short spans and bridge floors" and "main girders of long spans" quite separately. The definition of a "short" span for this purpose is given below.

The effect of the hammer blow is the most important and will now be considered.

6. *Effects of the Periodic Force.*—The weights introduced to balance the reciprocating parts are generally distributed over all or most of the driving wheels. These cause blows on the rail under each wheel. On very short

spans only one blow may be operative. For longer spans the sum of all the blows must be taken, that is, the resultant centrifugal force of all the balance weights considered as at the centre of this group. The effect is a series of blows delivered on the bridge at a regular and definite frequency proportional to the speed of the engine. The value of one blow varies as the square of the speed and in badly balanced engines may amount at high speeds to as much as 20 tons repeated several times a second. Any particular span has a definite frequency of vibration of its own, and it is not until the blows are delivered to the span at this frequency, or very near to it, that evil results. Then the effects of successive blows are cumulative and may be serious. It is under these conditions that the worst effects are produced. But the frequency of vibration of bridges increases as the span becomes less; and for any particular engine there are spans on which it can not, or in practice does not, reach the speed required to obtain this necessary synchronism. Then the conditions necessary to accumulative vibrations can not occur. This consideration sharply divides "short" and "long" spans from the point of view of impact. It lies generally between 50 and 75 feet spans, depending mostly on the diameter of the driving wheels; the smaller the wheel, the lower the dividing line. The periodic force causes impact on the "short" spans but in a different way.

THE COMMITTEE'S FORMULA AND ITS DERIVATION.

7. *Formula for Bridge Frequency.*—A study of the deflectometer diagrams of the tests made by the Indian Railway Bridge Committee in 1921, the Great Indian Peninsula Railway in 1922 and also the American Railway Engineering Association (*vide* Bulletin No. 125, where typical diagrams are given) shews clearly that the increase of central deflection under a train moving at speed is equal to the half amplitude of the vibrations set up about the mean or static deflection line. Further, the maximum impact or vibration is caused when the speed of the train, expressed in revolutions per second of the locomotive drivers, agrees with the frequency of vibration of the loaded span calculated by the following formula, *viz.* :—

$$n_0 = \frac{1}{\sqrt{\frac{(w+p)}{p} d}} \dots \dots \dots (1)$$

- where n_0 = natural frequency of vibration of the loaded girders.
 w = dead load per foot run of the span assumed to be uniform.
 p = equivalent live load per foot run of the train on the span at the position giving maximum bending moment.
 d = static deflection in feet corresponding the load p .

8. The practical accuracy of the above formula has been verified in the case of the Indian Railway Bridge Committee tests and a table is given (*vide* Sheet 2) from which it will be seen that the critical frequency calculated by formula and given in column (9) agrees closely with the observed critical frequency given in column (10). It is important to emphasize that the live load of the train has a fundamental effect in modifying the natural frequency of vibration of the unloaded girders and cannot be neglected. The above formula takes account of it.

9. *Typical Diagrams.*—Some typical examples of deflectometer diagrams are reproduced on Sheet 8 which illustrate the type of diagram and the cumulative vibrations obtained at critical speed. These diagrams are those giving the worst impact effects for the particular spans that they refer to. It may be noted that the amplitude of the vibrations in some cases goes on increasing after the point of maximum deflection has been reached, but actually we are not concerned with the magnitude of the vibration beyond the point of maximum deflection.

10. Since, as already stated, the vibrations correspond to the revolutions of the engine, it is evident that they are caused by a periodic force having the same frequency as the rotations of the drivers. The effects of periodic force have already been described in paragraph (6) and the method of calculating the "hammer blow" is given in Appendix A. For the purpose of comparing the periodic force of different engines the "hammer blow" is calculated at one revolution per second and is expressed by the symbol P_1 . At any other speed equal to " n " revolutions per second the "hammer blow" will be expressed by $P_1 n^2$. A table Sheet No. 7 is given shewing a comparison of the values of P_1 calculated for various engines in use on Indian Railways.

11. It would appear that the engine "hammer blow" must be a main controlling factor in producing the vibration, but the total value of the cumulative effect will depend on the span of the bridge and its frequency of vibration as controlled by the weight on it and the stiffness of the girders. In order to get some idea of how these various factors control and limit the possible impact, the following simple mathematical investigation was made. It is a simple synthetic computation and does not lay claim to represent dynamic laws or theoretical investigations of cumulative vibrations which tend to shew higher impact effects that are realized in practice on actual bridges.

12. **ADDED DEFLECTION.**—The question is, what is the *added deflection* on a span of length " L ", weighing " w " tons per foot run, due to a train weighing " p " tons per foot run, headed by an engine whose hammer blow is P_1 at one revolution per second, passing over the bridge at critical speed.

Let n_0 = critical frequency.

Then at every revolution of the locomotive drivers there is an upward and a downward impulse given to the bridge = $P_1 n_0^2$ (tons).

If there are " N " revolutions of the drivers in crossing the bridge, there are a total of $2N$ impulses counting both upward and downward as being cumulatively effective. The effect, however, of any particular impulse in producing central deflection depends on its position (*i.e.*, the loco's position) on the span. The effect will be quite small at the commencement, increase to a maximum at the centre of span, and then die away again. It may be taken as an approximation that the mean effect is equal to half the maximum. (Theoretically the mean is greater than one half but making allowance for damping, the factor of one half is closely representative of the actual.) Hence the total increase of central deflection due to $2N$ impulses is the same as that due to N impulses at the centre of the span. " N " is the total number of revolution in crossing the bridge, but as previously pointed out we are only interested in the number of revolutions up to the point of maximum deflection. From an examination of the diagrams of the Indian Railway Bridge Committee and the American tests it is apparent that on the average, the maximum deflection occurs when the engine has traversed about *two-thirds of the span*. Hence the number of cumulative impulses may be taken as $\frac{2}{3}$ of N .

We can now write down the added central deflection due to $\frac{2N}{3}$ impulses as—

$$\text{added deflection} = \frac{P_1 n_0^2 L^3}{48 EI} \times \frac{2N}{3} \dots\dots\dots (2)$$

Substituting for $N = \frac{L}{c}$ where c = circumference of driving wheel

$$\text{we have added deflection} = \frac{P_1 n_0^2 L^3}{48 EI} \times \frac{2L}{3c} \dots\dots\dots (3)$$

13. **Impact Formula.**—In order to express this as a fraction of the static deflection (*i.e.*, as an impact factor), we must calculate the static deflection and divide (3) by it.

The static deflection due to a distributed load of "p" tons per foot run may be taken to have an average value in ordinary girders given by the formula—

$$d = \frac{p L^4}{64 E I} \dots\dots\dots (4)$$

Hence we have the impact factor

$$\begin{aligned} i &= \frac{P_1 n_0^2 L^3 2 L}{48 E I 3 c} \times \frac{64 E I}{p L^4} \\ &= \frac{8 P_1 n_0^2}{9 c p} \dots\dots\dots (5) \end{aligned}$$

Reverting to equation (1) we can write

$$n_0^2 = \frac{P}{(w+p) d} \dots\dots\dots (6)$$

Substituting in equation (5) we have

$$\begin{aligned} i &= \frac{8 P_1 p}{9 c p (w+p) d} \\ &= \frac{8 P_1}{9 c (w+p) d} \dots\dots\dots (7) \end{aligned}$$

This represents the impact due to a single engine with hammer blow P_1 .

If we multiply by 100 to express it as a percentage, and express "d" in inches, we have approximately

$$\text{IMPACT PERCENTAGE} = \frac{1050 P_1}{(w+p) c d} \dots\dots\dots (8)$$

where P_1 = "hammer blow" at one rev. per sec. in tons.
 w = dead-load in tons per foot run.
 p = live-load in tons per foot run.
 c = circumference of loco. drivers in feet.
 d = static deflection in inches.

14. *Inferences from the Formula.*—The formula obtained above contains all the most important variables which affect impact due to engine synchronism and is believed to represent the true average of that effect realized in practice.

15. The formula shews that the impact effect on any bridge varies.

- (1) directly as the hammer blow of the engine at one revolution per second,
- (2) inversely as the circumference of the loco. drivers,
- (3) inversely as the total unit loading (dead load *plus* live load per foot run) on the bridge,
- (4) inversely as the static deflection of the girders in any particular case.

16. The length "L" of the span is generally considered to be a necessary ingredient of an impact formula. Although it actually does not occur in this formula, it is virtually present because it is a function of "d".

17. Factors (1), (2) and (3) are fairly straightforward. Factor (4) shews that the greater the static deflection *on any particular span*, under a

given load the less the impact. At first sight this may appear a rather startling statement, but its explanation lies in the fact, that the greater the static deflection the slower the rate of vibration of the span. This appears in formula

$$n_0 = \frac{1}{\sqrt{\frac{w+p}{p} \times d.}}$$

It follows therefore that the speed of the engine must be slower to obtain synchronism between the revolutions of the driver and the bridge vibrations. We have already seen that the periodic blow on the bridge is proportional to the square of the speed. Hence an increase of "d" decreases the critical speed and hence decreases the value of the periodic blow at critical speed in still greater proportion. See paragraph No. 6.

18. *Experimental Verification.*—In order to test the truth or otherwise of the formula as applied to actual bridges, advantage has been taken of available published experimental results where sufficient information has been given.

19. Three separate tables are attached to the report in the order in which they were worked out shewing a comparison of calculated impacts and maximum recorded impacts obtained in the following :—

- (i) Indian Railway Bridge Committee Tests, 1921. Sheet No. 3.
- (ii) American Railway Engineering Association Tests, 1908, published in Bulletin No. 125. Sheet No. 4.
- (iii) G. I. P. Railway Tests, 1922. Sheet No. 5.

20. In all these cases a close agreement is shewn between the recorded impact and the impact calculated by the formula. This can be seen by comparison of columns (3) and (10) in each Sheet.

21. In the case of the Indian Railway Bridge Committee Tests, 1921, the calculated impact is usually higher than the recorded excepting in the case of the two largest spans, viz., the 257 feet and the 358 feet where the calculated almost exactly agrees with the recorded. In the shorter span bridges in these tests it is presumed that the worst possible critical effects were not realized in the tests.

22. In the American Railway Engineering Association Tests, Sheet No. 4, there is agreement shewn between the recorded and calculated impact over a wide range of spans. In a few cases the calculated impact is less than the recorded, but it should be noted that the calculation depends on the information given regarding the value of the "out-of-balance" which fixes P_1 and there is some doubt in the case of engine No. 7027 as to whether the calculated hammer blow, which is extremely low, is correct. On the other hand the calculated value of the hammer blow of engine No. 939 is extremely high, and in that case the calculated impact is in excess of the recorded. This engine is a comparatively light one and the excessive value of P_1 is doubtful, hence the discrepancy. In those cases where the particulars given result in a calculated "hammer blow" of normal amount the agreement between observed and calculated impact is as near as in the nature of the problem can reasonably be expected.

23. The Great Indian Peninsula Railway Tests also shew close agreement.

24. *Case of engines of lighter "Hammer Blow."*—In order to provide a check on the formula as applied to the case of engines with light "hammer blow" Sheet No. 6 has been prepared which gives results of Indian Railway Bridge Committee Tests in 1920 and 1921. Although the value of P_1 for these engines is not much over 0.3 tons, i.e., about half the value of P_1 for the H. G., N. W. R. type used in the main tests, and the calculated impact

values are correspondingly low, there is very good agreement with the observed impact values, and in fact the calculated values are in all cases higher.

25. The conclusion is that the formula covers the case of engines with light hammer blow equally as well as the heavier, and the general conclusion is that the experimental evidence confirms the practical accuracy of the formula.

26. *Case of balanced compound and Electric Locomotives.*—The case of balanced compound and electric locomotives does not permit of experimental verification by the above formula. It is known that such locomotives do produce a certain amount of impact and the inference is that there is something analogous to a periodic force in the case of these engines. The general result of the American Railway Engineering Association Tests with engines of this type was that the impact on the whole was about one-third that of average steam locos. in those tests. The inference is that the value of P_1 will never be less than 0.2 tons for Broad Gauge engines.

27. *Utilization of the formula.*—It so happens that the formula gives a close agreement with facts not only in the longer spans but also in spans as low as 30 feet, although it is only strictly intended to apply to spans where actually synchronism is reached at ordinary speeds. The fact that high impacts are recorded on short spans at speeds below the critical speed indicates that the effects in those cases are not necessarily synchronous effects but are forced vibration effects.

28. There was some difference of opinion in the Committee as to whether these effects were covered by the formula or whether the latter was not excessive in the case of the very short spans. It was finally concluded that impact up to 100 per cent. may be reached on spans up to 20 feet and for spans above 20 feet the formula is applicable. [See para. 37 (b)].

29. *Case of existing Bridges, Broad Gauge.*—The synthetic or basic formula

$$i \% = \frac{1050 P_1}{(w+p) c d}$$

can be directly applied to the case of existing bridges and will give the impact that may be expected for any engine of which the value of P_1 is ascertained. The method of calculating P_1 is given in Appendix A. In the case of compound engines and electric locomotives the periodic force must either be determined experimentally or the value of P_1 assumed to be not less than 0.2 tons.

30. *Double-headed trains.*—Where double-headed trains are concerned the value of P_1 in the formula must be increased. It must be assumed that if two engines of the same type are exactly synchronised, their effect on spans of 120 feet and over will be double that of one engine.

For spans below 120 feet a factor must be introduced.

It may be taken—

(i) For spans of 120 feet and over

$$i \% = \frac{2100 P_1}{(w+p) c d}$$

(ii) For spans less than 120 feet

$$i \% = \frac{1050 P_1 \left\{ 1 + \frac{\text{Span}-30}{90} \right\}}{(w+p) c d}$$

31. If two engines of different diameters of driving wheel are used, it is reasonable to suppose that they can never give more than the impact of one engine. This is a special case which probably requires further investigation.

32. *General Formula for New Bridges, Broad Gauge.*—In designing a bridge to a given standard of loading, we have to decide what is the out-of-balance “hammer blow” that must be allowed for. A reference to Sheet No. 7 shews the value of P_1 for some typical engines on Indian Railways, and it would appear that the H. G., 2-8-0, N. W. R. type has the highest value of P_1 . Further it so happens that a double-headed train with this type of engine and wagons weighing about 1.35 tons per foot run is approximately equal to Standard II loading. It is therefore reasonable to take this engine as a criterion of the worst case to be covered by this standard.

33. In order to construct a general covering formula, it is necessary in the first instance to consider the impact in relation to the total live load in question, as distinct from the impact due to a particular test load. The impact value of a given engine obtained in a test may be a high percentage of the engine weight but the latter may be only a fraction of the capacity load of the bridge. It is therefore important, if we express impact as a percentage of the capacity load, that the value so obtained shall not be greater than the effect of the two engines which are allowed for.

34. It has been shewn that

$$i \% = \frac{1050 P_1}{(w+p) c d} \text{ for single engines.}$$

This formula is equally true if we substitute for test values of “p” and “d” the values “ $p_{\max.}$ ” and “ $d_{\max.}$ ” for the capacity load of the bridge.

Hence we have

Impact on capacity load for single engine trains

$$= \frac{1050 P_1}{(w+p_{\max.}) c d_{\max.}}$$

35. In the case of Standard II loading the following values may be substituted for the above variables:—

(1) It can be shewn that $(w+p_{\max.}) = 3.0$, very closely
for nearly all spans (under new impact allowance).

(2) $P_1 = 0.604$

(3) $c = 14.8'$

(4) Taking $d_{\max.} =$ one two thousandth of span
 $= \frac{L \times 12}{2000}$

$$\begin{aligned} \text{We have } i \% &= \frac{1050 \times 0.604 \times 2000}{3 \times 14.8 \times L \times 12} \\ &= \frac{370}{L} \end{aligned}$$

36. This has been plotted and is shewn by curve A on Sheet No. 10 which represents the maximum calculated effect of a single H. G. engined train. To verify this Sheet No. 9 has been prepared giving the calculated impact percentage on full capacity loads for various spans tested in the Indian Railway Bridge Committee, American Railway Engineering Association and Great Indian Peninsula Railway Tests. The values obtained in column (9), Sheet No. 9, have been plotted on Sheet No. 10 and the values fall very closely on the above curve.

Hence curve A can be taken as closely representing impact for single engines. Curve B has next been added to represent the impact of double engines on the basis of:—

$$(1) \text{ Spans of 120 feet and over } i \% = \frac{4740}{L}$$

$$(2) \text{ Spans below 120 feet } i \% = \frac{2370}{L} \left\{ 1 + \frac{\text{Span} - 30}{90} \right\}.$$

Finally a covering curve for general impact is drawn to the formula:—

$$\text{IMPACT FACTOR} = \frac{65}{45 + L}$$

where L = loaded length in feet.

37. *Other causes of Impact.*—In paragraphs 4 (ii) and 4 (iii) reference is made to rolling load effect and to irregularities of track, etc., respectively, as causes of impact.

The Committee are of opinion that there is very little evidence on this subject as regards Indian railway conditions on which to form a definite opinion as to its quantitative value and importance.

At the same time certain conclusions can be drawn which may be stated as follows:—

- (a) *Rolling Load Effect.*—The evidence of tests made with engines on bridges under service conditions does not indicate that there is any important effect from this cause requiring any addition to the engine effect.
- (b) *Irregularities of Track, Flats on Wheels, etc.*—As regards irregularities of track, in general it may be concluded that the tests do not provide evidence, excepting in the case of rail bearers, of special impact from this cause over and above the engine effects. In the case of short spans, further evidence seems desirable before giving a quantitative opinion for application to existing bridges. There is some evidence that a further reduction of impact, applying to short spans, may be possible if legislation for elimination of rail joints on short spans could be resorted to.

As regards flats on wheels it does not seem reasonable to suppose that a synchronous effect from this cause could be worse than the effect of the engine ‘hammer blow’ for which adequate allowance has been made. In any case its effect except in the rarest cases is unlikely to be an additive one.

H. N. COLAM.

F. HARRINGTON.

L. H. SWAIN.

MUZAFFAR HUSSAIN.

APPENDIX A.

METHOD OF CALCULATING THE HAMMER BLOW.

The resultant effect of the overbalance weights on all wheels when the engine runs at one revolution per second has been expressed by the Symbol P_1 . In any given engine this can be calculated if we know the equivalent out-of-balance weight at the crank pin. The equivalent out-of-balance weight at the crank pin is represented by M and is a percentage of the total of the reciprocating masses on one side of the engine. This percentage varies from $66\frac{2}{3}$ to only 45 per cent. depending on the practice adopted.

The reciprocating parts are :—

Piston and rod.

Crosshead.

Small end of connecting rod (*i.e.*, proportion of whole considered as reciprocating).

If the weights of these are known and the percentage balanced is also known the value of M can be determined.

Let r = radius of crank pin in inches.

Then the centrifugal force of the equivalent weight M rotating at one revolution per second at radius r is equal to :—

$$\frac{M (2 \pi r)^2}{g \times r \times 12}$$

$$= \frac{\pi^2 M r}{g \times 3}$$

This represents the centrifugal force of the balance weights on one side of the engine. Since the weights on the other side are at 90° the resultant effect from both sides is equal to :—

$$\sqrt{2} \times \frac{\pi^2}{g \times 3} \times M r$$

$$= .145 M r \text{ lbs.}$$

Or expressed in tons

$$P_1 = \frac{.145 M r}{2240}$$

$$= .000065 M r$$

APPENDIX B.

AMERICAN RAILWAY ENGINEERING ASSOCIATION TESTS.

Comparison of Impact obtained from stress recorders, deflectometers and by calculation.

Span in feet.	Test series	IMPACT PERCENTAGE.			Ratio. <u>Column (3)</u> <u>Column (4).</u>	Ratio. <u>Column (3)</u> <u>Column (5).</u>
		RECORDED.		Calculated by formula.		
		Stress.*	Deflection.			
(1)	(2)	(3)	(4)	(5)	(6)	(7)
50	1008	74	67	78.6	1.10	0.94
60	1074	43	40	60	1.08	0.72
65' 5"	673	57	48	49.4	1.18	1.15
80' 0"	430	56	44	43.4	1.27	1.29
80' 0"	660	44	37	77	1.19	0.57
92' 7"	717	37	39	41	0.95	0.90
100	1042	37	46	43	0.80	0.86
124	1000	38.5	36	59	1.07	0.65
132	277	38	31	25	1.23	1.52
210	—	25	24	30	1.04	0.84
300	—	20	22	22	0.91	0.91

* Selected values of results considered reliable.

Averages—

(a) Mean value of column (6) —

$\frac{\text{observed stress impact}}{\text{observed deflection impact}}$

= 1.08

(b) Mean value of column (7) —

$\frac{\text{observed stress impact}}{\text{calculated impact}}$

= 0.94

APPENDIX C.

IMPACT FORMULA FOR STANDARD III B. G. DESIGN.

(i) The Impact for Standard III depends on the estimated value of the hammer blow in locomotives yet to be designed. A reference to the Director of Mechanical Engineering elicited the opinion that P_1 can "be kept within .034 of the weight of the main driver." Standard III locos have four 24-ton axles, giving a value of $P_1 = .82$ ton. Dividing by the corresponding value of the driving wheel circumference we obtain value for $\frac{P_1}{c}$ of about .056.

(ii) As a matter of interest it is pointed out that if the particulars of existing locos be examined it is found that the value of $\frac{P_1}{c}$ divided by the total weight of *all* the drivers tends to be constant, and about .00065. This indicates that $\frac{P_1}{c}$ varies as the total adhesive weight, not as the weight of the main driver only. In other words the value of $\frac{P_1}{c}$ varies as the power of the engine.

(iii) The value of $w + p$ (i.e., the total load per foot run) for Standard III for spans 60 feet up to 300 feet is almost exactly 4.0 tons. Using this value, and one two thousandth of the span for the deflection, the Impact for one engine = $\frac{1050 \times .056 \times 2000}{4 \times L \times 12} = \frac{2450}{L}$. This is so near the value obtained for Standard II that it is simpler to use the same formula for both, namely, $\frac{2370}{L}$. This means that the maximum permissible value for $\frac{P_1}{c}$ for Standard III engine is .0542 ton and this should be specified as the limit for the guidance of loco designers.

APPENDIX D.

IMPACT ALLOWANCE ON MULTIPLE-TRACK BRIDGES.

Broad gauge.

(a) *Design*.—The impact tests hitherto recorded have been made with one train only and there are no available results for bridges tested with trains on all tracks simultaneously.

Hitherto the practice in India has been to calculate the full impact effect on each track separately, and no reduction has been made for additional tracks. In this connection, however, it is reasonable to consider the probability of exact synchronism occurring on all tracks simultaneously.

The impact formula for design for single track bridges is $\frac{65}{45+L}$. This covers the case of a double-headed train with both engines synchronized. In the case of multiple-tracks it is extremely improbable that the condition would be fulfilled on all tracks. Furthermore, it is still more highly improbable that four engines (in the case of a double track bridge) should be exactly synchronized in regard to speed, phase of counterweights, and that they should arrive on the span simultaneously.

It is therefore reasonable to make a reduction of impact in multiple-track bridges. In the B.E.S.A. Specification it takes the form of $I = \frac{120}{90 + \frac{n+1}{2}L}$ where “n” equals the number of tracks. This type of formula, however, does not give a proportionately greater reduction of impact in small spans.

The Committee propose the following formula, *for main girders only* which gives a greater reduction on short spans where the possibility of synchronism of two trains is less than on long spans, *viz* :—

Impact Factor = $\frac{65}{45n + L}$ where “n” equals the number of tracks supported on two girders.

The case of cross girders and rail bearers requires full impact on all tracks.

(b) *Existing Bridges*.—Cases of existing bridges can be dealt with on their merits by the application of the basic formula.

$$i \% = 2 \times \frac{1050 P_1}{(w + p)cd}$$

which covers either the effect of two single engined trains synchronized on the two tracks or the effect of a double headed train on one track.

APPENDIX E.

" SECOND CRITICAL SPEED. "

I. R. B. C. Tests, 1921.

Mr. Lloyd Jones made special reference to the tests on the Adamwahan 257 feet span and the Kotri 358 feet span. In these bridges there are undoubtedly, at a speed of double the critical speed, bridge vibrations shewn on the deflectometer diagrams of appreciable amount, though less than the vibrations at the critical speed. Examination, however, of the diagrams shews clearly that the cumulative vibrations (see diagrams reproduced on Sheet No. 12) are of the same frequency as the critical frequency and not at double that frequency, which would be the case if they were synchronous with the engine. There is moreover evidence to show that a long span, if it vibrates at all, will vibrate invariably at its proper critical frequency irrespective of the train speed. Hence there is no evidence of a second synchronization. The explanation of the fact that there is a certain cumulative effect when the train goes at double critical speed probably lies in the fact that the bridge gets a synchronous hammer blow at every other revolution of the locomotive driving wheel which blow does not get completely damped out by the subsequent blow, and manages to become cumulative. In no case however is the cumulative effect at double critical speed as great as the effect at critical speed. In this connection a comparison can be made between diagram No. 1426 on sheet No. 12 with diagram No. 1440 on sheet 8. These are for the Kotri bridge. The former is a deflectometer diagram at double critical speed and the latter at critical. A similar comparison can be made of diagrams Nos. 958, 962 and 1067 on sheet No. 12 with diagram No. 938 on sheet 8, being diagrams for the Adamwahan bridge.

The Committee are of opinion that the alleged " second synchronization " is not realised in practice and they believe that the critical speed proper corresponding to the natural period of vibration of the girders about their supports is the true and essential criterion which regulates impact from engine synchronism.

APPENDIX F.

CASE OF METRE GAUGE BRIDGES.

The Metre Gauge Engines of which we have particulars of balancing give:—

Class.	Type.	Adhesive weight in tons.	M. lbs.	R. inches.	P ₁ tons.	C. ft.	$\frac{P_1}{c}$	$\frac{P_1}{cw}$
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Rendel, Palmer and Tritton's letter of 5th March, 1925.

R	4-6-0	25.7	274	11	.186	12.6	.0148	.00057
P	4-6-0	26.1	295	11	.21	14.9	.0142	.00054
Q	4-8-0	31.85	312	11	.223	11.0	.0203	.00064

Obtained from Tritton by Mr. Everall, M. & S. M. Railway Engines.

P ₆	4-6-0	27.7	277	11	.198	14.9	.0134	.00049
G ₆	4-8-0	34.4	275	11	.196	11.3	.0174	.00051
M	4-6-0	25.2	268	11	.192	12.6	.0152	.00061

B. B. & C. I. Railway letter, April 6th, 1925.

P	4-6-0	28.4	211	11	.15	14.9	.0101	.00036
G ₂	4-6-0	27.8	184.5	11	.132	12.6	.0105	.00038
F ₂	0-6-0	23.9	172	11	.123	11.3	.0109	.00045

In the Bombay, Baroda and Central India Railway engines only 50 per cent. of the reciprocating parts is balanced as against 66 per cent. in the others. This accounts for the low value of $\frac{P_1}{c}$ for these engines. Presumably the balancing of these engines is satisfactory as they have been running for years, and if this is so there seems to be no valid reason why engines of the future should not have correspondingly low values of $\frac{P_1}{c}$.

If then we take $\frac{P_1}{c_w}$ as .0004, the value of $\frac{P_1}{c}$ for a Standard III M. G. engine with four 15 ton drivers is $60 \times .0004 = .024$ and the Impact Factor for an engine = $\frac{1050 \times .024}{2.95 \times 1.1 \times 12} \times 2000 = \frac{1420}{L}$ and for two engines = $\frac{2840}{L}$.

If the worst balanced present day engines be adopted as the criterion $\frac{P_1}{c}$ for Standard III is $60 \times .00064 = .038$ and the Impact Factor $\frac{2250}{L}$ which is practically the same as that adopted for the B. G.

The Impact Factor for the M. G. cannot be definitely established until it has been confirmed by experiment.

APPENDIX G.

BEARING STRESS OF RIVETS IN OLD WROUGHT IRON GIRDERS.

The following are particulars of the bearing stresses on a number of old Iron M. G. girders on the Madras and Southern Mahratta Railway calculated by the Burma formula and by the B. E. S. A. formula. Pencoyd Impact has been allowed, and the load is the F. M. Engine which has been running on these girders for a great number of years. Due allowance has also been made for the local effect of the wheel load. There are no loose rivets in these girders.

The webs were carefully measured, but it is just possible that where the web is exposed, and where of course it was measured there may have been some slight loss by rusting, and that where the web is protected by the angle it is somewhat thicker. For this reason $1/16''$ is added to the measured thickness of web and the stresses recalculated, and tabulated alongside the stresses calculated from the measured thickness:—

Span.	Drawing No.	BEARING STRESS.			
		By BURMA FORMULA.		By B. E. S. A. FORMULA.	
		On measured web.	On web $+1/16''$.	On measured web.	On web $+1/16''$.
8'	115 M. G.	10.7	8.9	14	11.7
9'	15 M. G.	13.2	10.6	16.9	13.5
10'	53 M. G.	11.4	11.4	15.2	15.2
10'	94 M. G.	11.5	9.6	15.4	12.8
12'	38 M. G.	15	12.5	18.8	15.6
20'	103 M. G.	12.1	10.1	15.6	13
20'	110 M. G.	15	12.0	16.5	13.2
20'	111 M. G.	14.2	11.3	15.5	12.3

VI

12-14, DARTMOUTH STREET,
WESTMINSTER,
LONDON, S. W.-1,
28TH MAY, 1925.

DEAR SIR,

Report on the Government of India Rules for the Inspection of Railway Bridges and the Testing of Girders.

We have your letter dated 22nd December, 1924, enclosing Terms of Reference proposed for the Committee which will shortly be assembled to examine and report on the Government of India Rules for the Inspection of Railway Bridges and the testing of Girders.

2. We have had many of the points raised in the Terms of Reference under consideration since the 1923 Bridge Rules were issued, with the intention of bringing them before you. Now that you yourself have brought them before us, we will make our suggestions in conformity with the manner in which you introduce them.

3. It is to be regretted that the Indian Railway Bridge Committee, convened in 1917, did not continue their bridge testing, including the determination of secondary stresses in the various members of the different types of bridges which were tested for Impact Effect, for such stresses are of great importance.

4. It was also unfortunate that the test load used during the Indian tests was not the maximum permissible and that each bridge was not loaded from end to end, as well as by a standard load of one locomotive independent of the span length. The test loads used for the Ministry of Transport tests, carried out by ourselves, were the heaviest engines obtainable, and, where the length of the span required it, two coupled engines were employed and the greatest possible speed was obtained—on one railway as great as 84 miles per hour. We can therefore say that the approximate maximum effect of the live load was obtained in every test. ⁽¹⁾ Under the circumstances, we think it would have been advisable to have accepted the B.E.S.A. Formula entirely, instead of up to 50 feet only—the use of the Pencoyd Formula above 50 feet requires, according to recent tests, the use of more steel than seems to be necessary. We note with satisfaction under II—Duties of Inspectors—page 5 of the new 1923 Rules, that all bridges are to be tested with two of the heaviest locomotives, and that, if these do not cover the length of the bridge, the remainder is to be loaded with wagons of the heaviest type. We also note that, during the tests to be made, deflection tests are specified. We consider that these tests are very important, and will be more so in the future.

5. We should like to point out that no particulars of any records taken during these tests in India have yet been sent to us, although full particulars were sent immediately to the British Engineering Standards Association, and we cannot help feeling that this omission must have been due to some misunderstanding. We, however, obtained the reports from the B.E.S.A. and went very fully into the tests, obtaining valuable and new information. No one in this Country but ourselves could have analysed the tests, because only we have the detail drawings and the original calculations of the bridges tested. The B.E.S.A. were therefore not in a position to make use of the particulars sent to them.

6. You are aware that the Bridge Stress Committee of the Department of Scientific and Industrial Research are carrying out bridge tests on a large scale in this Country. The Term of Reference to this Committee was, at first, Impact only, but we understand that the determination of secondary stresses was included afterwards. If you are not prepared to wait until this Bridge Stress Committee have completed their work and issued their final report, which will certainly be another year or more, then, perhaps, the work you propose to do of making a thorough analysis of the weights of girders designed under different Rules, although very laborious, will

(1) As all the Bridges were double track, they were none of them fully loaded.
(L. E. H.)

throw some light on the subject, but it appears to us that the comparison of the weight of two bridges, of the same length and carrying the same load, designed under different bridge rules would not be conclusive without the comparison of secondary stresses. In addition to this proposed investigation, we suggest in paragraph 15 that further tests might be carried out on the same lines as suggested from time to time by Mr. Lloyd-Jones.

7. There is a very wide field in India for further scientific analysis of bridges for the determination of secondary stresses. Much has already been done, and, while the main idea was the determination of such stresses, the investigations have ended in saving bridges which have been marked down for replacement.

8. Mr. Lloyd-Jones' Report on his tour in America (Technical Paper No. 225), is a very valuable contribution on the subject—we know of no one more qualified to have undertaken the work and to have drawn useful conclusions. The various opinions expressed by the leading Engineers in America, and the American Bridge Rules, show the great difference there is between the practical consideration and the mathematical consideration as given in some of the American text books.

9. With regard to the Terms of Reference to the Bridge Committee, we suggest that they should be altered to read as follows:—

“ The object of the enquiry is to ascertain, as rapidly as possible, in what way and to what extent the cost of new steel bridges in India (i) for railway trains, (ii) for roadways and (iii) for a combination of both these loads, can be reduced. ”

10. We are in close agreement with the Committee's endeavour to reduce the cost of steel bridges in India. We have always given this matter the closest attention, and are giving effect to it, first by reducing the weight of steel in the bridges to a minimum while complying with the Indian Bridge Rules, by the standardisation of plate girders, from 6 to 80 feet spans, for the different loadings and gauges, and of trusses from 100 to 250 feet spans, and in all cases so to make the designs that by increasing the number of similar parts and by simplifying all the details, a contract for 10 or more similar spans of almost any type of bridge can be executed, under proper supervision, with the highest possible quality of work made perfectly interchangeable by the use of steel bushed jigs, at a price *less* than for the ordinary class of bridgework. In the latter class of work, every span has to be erected in the Contractor's Works and every part match-marked to its place, while, in the new method, only a span now and then is erected and no match-marking is required. As all the similar parts in the spans are interchangeable, erection at Site is simplified and consequently cheapened.

It may be of interest to you to know that nearly every Contract in this Country for ten or more similar spans for India is made with firms who make this interchangeable work by steel bushed jigs, because they can do it at about 15 per ton cheaper than the ordinary class of bridgework. Moreover, there is gradually being accumulated by one firm or another complete sets of steel templates for *Standard Spans*, for Indian Railways, which must have an effect on future prices.

11. We consider that the investigations being carried out in order to produce a new impact formula will probably result in some slight reduction in the allowance for Impact Effect on long spans, but that the real reduction in the weight of steel will come later by an increase in the maximum permissible stress of the steel itself.

It is obvious that the testing of bridges which has taken place in many countries during the last fifteen years, coupled with the more scientific methods of determining the stresses in structures, must lead to more complete knowledge of what the *maximum* stresses are under the effect of live load and this should make it permissible to stress the material nearer to the limits of proportionality. It must not be forgotten, however, that with the use of steel having a definite modulus of elasticity, the higher the stress per square inch, the greater are the secondary stresses. The secondary stresses in any structure can be kept within limits, and almost eliminated, by good design, by certain methods of manufacture and by a particular

APPENDIX.

During the Impact Experiments made by the Indian Railway Bridge Committee on the North Western Railway between January and April 1921, the reflections at the centres of all the 10 spans tested were recorded under dead slow speed and higher speeds, and under different classes of locomotives. As we have the drawings of all these spans here, we calculated the maximum deflection at the centre of all the spans under the locomotives used, and by the most exact method, *i.e.*, "The Principle of Least Work." In every case, the calculations gave a higher result than the actual, the ratio of $\frac{\text{actual}}{\text{calculated}}$ varying from about .75 to .87.

In some of the truss spans, notably the East Beyne Bridge and the Sutlej Bridge, we went further than that, and we took the extensometer records of the D. S. tests, where the stress recorders were placed approximately in the neutral axis of the members, and from these stress records, we calculated the stress in the whole member as represented by the stress record. We compared these results with the figures on the original stress sheets, the calculations on those stress sheets having been made by Rigid Body Statics. In obtaining the stress in the member, we multiplied the average cross section of the member (*i.e.*, with the rivet holes deducted) within the range of 20 inches (being the length of the instrument) by the fibre stress recorded in the instrument. We naturally found that in every case the calculations of the stress in the member performed by Rigid Body Statics were in excess of the actual as deduced from the instrument. The actual deflection recorded we accept as being correct. The calculations of the deflection as we do them are not exact because Rigid Body Statics have to be used in their determination, which are not applicable to elastic structures. These calculations are based on the theory that the whole of the work done in deformation is axial and there is no account taken of the deformation due to bending of the members and the work done in doing the bending, nor is there any relief of stress taken in the stresses from the floor system.

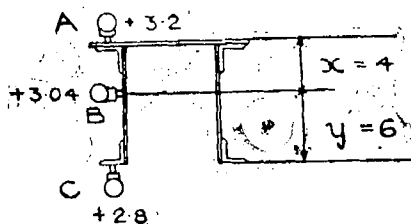
We found in every case that if the stress, as calculated by Rigid Body Statics, which appear on the original stress sheets, be multiplied by the ratio of the actual deflection and divided by the calculated deflection, the result is in very close agreement with the actual stress as determined from the instrument. We enclose herewith Sketches Nos. 523 and 524 showing the comparison.

We have found on several occasions that an attempt is made to compare the fibre stress recorded with the net section of a tension member, the net section in such cases being determined by one of the numerous Rules for doing so, varying in amount accordingly. Such comparisons cannot be made with accuracy and we therefore give here shortly the methods which should be employed in comparing, where necessary, the stress recorded with the stress as originally calculated.

1. The fibre stress recorded by the instrument is tons per square inch on the average section of the member between the points of the instrument, *i.e.*, 20 inches.
2. To ascertain the average section in a tension member, the volume of the member between the points of the instrument *minus* the total volume of the rivet holes must be found. This volume divided by 20, will give the average cross section. The average cross section of a compression member is the gross section, assuming the rivets are all tight.
3. To obtain the total stress in the member, the recorded fibre stress must be multiplied by the average cross section.
4. To obtain the *maximum* stress per square inch in any part of the member, the total stress in the member must be divided by the *net* section at that part.

Method of checking the axial fibre stress when obtaining secondary stresses by simultaneous readings from two stress recorders.

For example :—



AB & C ARE THE FIBRE STRESSES PER SQ. INCH RECORDED IN THE CORRESPONDING INSTRUMENTS

$$B = \frac{Ay + Cx}{x + y}$$

$$= \frac{(3.2 \times 6) + (2.8 \times 4)}{4 + 6} = 3.04$$

REF. MESSRS. RENDEL PALMER & TRITTON'S LETTER DATED 28TH MAY 1925,
IN REPLY TO THE TERMS OF REFERENCE.

Since this letter was written the Interim Report of Sub-Committee has been published, Messrs. R. P. & T. have seen and since replied to that provisionally in their letter dated 28th July 1925, to the Secretary, Standing Committee of Chief Engineers. The Sub-Committee's comments on that letter were put up recently.

It may be taken that the present reference so far as it deals with impact and Bridge Rules has already been disposed of.

The question of secondary Stresses has been dealt with in the Committee's redraft of the Rules, and they have separated deformation stresses, which latter they consider, in normal cases, are not of great importance.

The Consulting Engineer's remarks on the subject of standardization are of interest and shew the progress that has been made in the standardization of details as well as designs as a whole. Apparently details are now made interchangeable.

The question of adopting the British Standard Unit Loadings which is contemplated in the wording of the present Bridge Rules and to which the Consulting Engineer's object, is ruled out in the Committee's redraft of the Rules, and the objection is therefore answered.

The Loco. Standards Committee have accepted the figure for engine balancing, *vide* copy of letter dated 28th July 1925, attached.

H. N. COLAM.

F. HARRINGTON.

L. H. SWAIN.

31st August, 1925.

12-14, DARTMOUTH STREET,

WESTMINSTER,

LONDON, S. W. 1,

28TH JULY, 1925.

Interim Report of Bridge Sub-Committee.

DEAR SIR,

We have your letter of the 17th June sending us a copy of the Interim Report of the Bridge Sub-Committee, together with a complete set of Drawings. We are looking carefully through these papers, and will let you have our considered remarks as soon as possible.

2. We might, however, say at once, that the assumptions on which the proposed Impact Formula is based are somewhat too general for obtaining a reliable formula. For instance, Equation No. 2, page 5, is the deflection for a beam of uniform *section*, but the designer of a bridge always does his best to make it of uniform *strength*, in which case, the factor of 48 in the denominator would be nearer 32. Equation No. 4, however, is based on the fact that the girder is of uniform strength, and therefore Equation No. 5 would be nearer $\frac{4 P_1 n^3}{3 c p}$.

Again, the Committee take the magnitude of an impulse from the unbalanced effect of driving wheels as the *maximum* value of the centrifugal force. The actual impulse is a varying one, and if the simplification of substituting a constant impulse equal to the average centrifugal force is introduced, Equation No. 8 would read—

$$\text{Impact Percentage} = \frac{1000 P_1}{(w+p) c d}$$

which agrees fairly well with $\frac{1050 P_1}{(w+p) c d}$ given in the Report, but it is based on two assumptions, both different to those used by the I. R. B. C. It is a fortunate coincidence that the two corrections practically cancel each other.

The I in Equations 2, 3, 4 and 5 is hypothetical for the general case of a girder of uniform strength. I—is at the centre of a girder and the girder is of uniform depth.

3. We agree with Clause 17, but it opens up another question as to what value should be assigned to d in devising a formula for impact effect, on bridges of all spans where the basis is the synchronous speed of the train. The formula proposed is a covering one, based on a deflection of $\frac{1}{2000}$ of the span, but it might be worth while to consider a modifying factor of some constant of $\frac{L}{D}$ —instead. Special cases occur where the ratio of depth over length of span is less than that on which $d = \frac{1}{2000}$ is based. The effect of this would be to reduce the value of impact effect in such cases and would go to balance the loss due to an uneconomic depth.

4. With regard to Equation No. 1, from which the natural frequency of the bridge is determined, it should not be forgotten that p is supposed to be uniformly distributed as well as w, but in many of the spans tested in India the length of the test locomotive was a comparatively small factor of the length of the span, and it would not be true to reduce this load to an equivalent uniformly distributed load in the equation. For the equation to be correct, p should be taken as a distributed load which will give at all points a similar form of deflection curve with a deflection of the centre equal to d.

5. The above are a few of our criticisms regarding the assumptions on which the formula is built up. We think that further bridge tests might be taken to obtain evidence, from which corrections might be made to the

different formulæ, which are only true under ideal conditions, and these rarely exist in bridgework.

Yours faithfully,

(Sgd.) RENDEL PALMER & TRITTON.

The Secretary,
Standing Committee of Chief Engineers,
Simla, India.

COMMITTEE'S NOTE ON LETTER DATED 28TH JULY, 1925, FROM CONSULTING ENGINEERS, TO THE SECRETARY, STANDING COMMITTEE OF CHIEF ENGINEERS, SIMLA.

This letter accepts the principles on which the formula is based, but criticizes the details of its construction. It must be realised that in the nature of the problem any attempt at exact evaluation of impact or a formula which will apply to all cases with precise accuracy is impossible.

The Consulting Engineers suggest certain refinements in the formula.

The refinements they suggest are admittedly correct in principle, but it is not clear to the Committee that any great benefit is to be derived by such refinements.

Regarding paragraph (2) the difference between the constant 1050 given by the Committee and 1000 obtained by the Consulting Engineers is unimportant.

Regarding paragraph (3), by taking " d " = $\frac{L}{2000}$, they have calculated a higher impact than would be the case if " d " be taken as some other proportion such as $\frac{L}{1200}$, which might apply in the case of shallow girders. In the latter case a lower impact value could be taken but a further reduction hardly seems necessary. The general idea of the Committee has been to cover the worst cases by a covering formula.

Regarding paragraph (4), a reference to sheet No. 2, accompanying the Interim Report will shew that the calculated and observed critical speeds in the Indian Tests agree closely and the refinement suggested is therefore unnecessary.

As regards paragraph (5), the Committee so far from regarding the formula as only being true for ideal conditions, believe it to be essentially a practical formula which is confirmed by experimental evidence.

VII

CORRESPONDENCE WITH THE SECRETARY, LOCO. STANDARDS COMMITTEE ON THE BALANCING OF ENGINES.

COPY OF LETTER NO. 8/B. COM., DATED THE 5TH MAY, 1925, FROM THE SECRETARY, STANDING COMMITTEE OF CHIEF ENGINEERS, SIMLA, TO THE SECRETARY, LOCO. COMMITTEE, SIMLA.

The Bridge Sub-Committee require some information about Engines. The points under discussion are herewith enclosed. It will greatly help the deliberations of the Bridge Committee if this information could be furnished to them at the earliest opportunity.

The formula for impact derived by the Committee contains a quantity P_1 which is the hammer blow on the rail due to the defect from perfect balance in the engine.

It is vital to decide whether this quantity will increase in the future in proportion to the increase in the weight of the engines or in some less proportion.

The hammer blows of existing types of engines have been worked out and are tabulated here. In view of the fact that more attention is likely to be paid to the question of balancing in the future than in the past, is it necessary to provide for a worse hammer blow than is given by the H. G., 2-8-0, N. W. Railway engine?

The answer to this question must consider engines at least as heavy as the engines of the B. B. & C. I. Railway 1916 standard and loading, *i.e.*, a passenger engine with two 28-ton axles or a goods engine with four 24-ton axles.

If a quantitative reply cannot be given at once, a provisional expression of opinion on broad lines would be extremely valuable.

COPY OF LETTER NO. 1-L. S. C., DATED THE 6TH MAY, 1925, FROM THE SECRETARY, LOCO. STANDARDS COMMITTEE, SIMLA, TO THE SECRETARY, STANDING COMMITTEE OF CHIEF ENGINEERS, SIMLA.

Your No. 8-B./Com., dated 5th May, 1925.

It is considered by my Committee that in the case of future locomotives the total hammer blow can be kept within *0.34 ton per ton of weight on driving wheels*. It is very improbable that the weight on drivers will exceed 22.5 tons within the next fifteen years but provision should be made for an eventual axle load of 26.9 tons with a wheel arrangement similar to that shown on L. S. C. Drawing No. 15 of the Report of Loco. Standards Committee, 1924.

FROM SECRETARY, BRIDGE SUB-COMMITTEE, TO SECRETARY, LOCO. STANDARDS COMMITTEE, DATED SIMLA, THE 16TH JULY, 1925.

(1) With further reference to your No. 1-L. S. C., dated May 6th, 1925, and to conversation between the D. M. E. and yourself and the members of the Bridge Sub-Committee, I enclose particulars of balancing of engines asked for.

(2) The Sub-Committee wish to limit the out-of-balance of Standard III engines as follows:—

$$\frac{Mr}{c} = 830 \text{ Maximum.}$$

where M = weight in lbs. of balanced portions of reciprocating masses at one side of engine.

r = radius of crank pin in inches.

c = circumference of drivers in feet.

(3) The figure is confirmed as follows on the basis of the B. B. & C. I. M. type engine, *viz.* :—

$$4-19\cdot5\text{-ton axles, constant} = \frac{640 \times 14}{14\cdot8} = 605.$$

By analogy,

$$4-22\cdot5\text{-ton axles, constant} = \frac{22\cdot5}{19\cdot5} \times 605 = 700.$$

$$4-24\text{-ton axles, constant} = \frac{19\cdot5}{24} \times 605 = 745.$$

(4) The figure 830 was obtained from your original figure for $P_1 = \cdot034$ per ton of axle applied to a 24-ton axle engine and our impact curves are based on it and have been accepted by the Chief Engineers. Your formal acceptance and confirmation is kindly requested.

COPY OF LETTER FROM THE SECRETARY, LOCO. STANDARDS COMMITTEE, SIMLA, TO THE SECRETARY, BRIDGE SUB-COMMITTEE, RAILWAY BOARD, SIMLA, No. 1-L. S. C., DATED THE 29TH JULY, 1925.

Limitation of the "out-of-balance weight" of locomotives.

Your letter No. 8-B./Com., dated 16th July, 1925, was placed before the Loco. Standards Committee at the meeting held on the 28th instant, and I am instructed to inform you that the Committee agreed that the out-of-balance weight of new standard engines can and will in future be kept within the limit of $\frac{M r}{c} = 830$.



VIII

To

THE DIRECTOR OF CIVIL ENGINEERING.

SIMLA;

Dated September 4th, 1925.

DEAR SIR,

The Bridge Sub-Committee herewith submit their 2nd Interim Report on the work done to date on the subjects of " Impact " and " Revision of the Bridge Rules ".

Yours faithfully,

H. N. COLAM,

F. HARRINGTON,

L. H. SWAIN,

Members, Bridge Sub-Committee.

MUZAFFAR HUSSAIN,

Secretary, Bridge Sub-Committee.

IX

BRIDGE SUB-COMMITTEE, 1925.

Second Interim Report—Subject II—Proposed Revision of the Bridge Rules.

(1) The Bridge Sub-Committee have been through the Bridge Rules of 1923 and made what alterations they think necessary. The 1923 Rules were divided into eight sections:—

- (a) General Rules applicable to all New Bridges.
 - I. Duties of Senior Government Inspectors.
 - II. Loads and external forces for which stresses must be calculated.
 - III. Standards of train load and other external forces.
- (b) Special Rules for steel and iron bridges.
- (c) Special Rules for timber girders and trestles.
- (d) Special Rules for arched masonry bridges.
- (e) Special Rules for ferro-concrete bridges.
- (f) Applicability of the Rules to existing bridges.

(2) The Committee have redrafted Section (a) II entirely, following the B. E. S. A. Specification with some alterations, the most important of which are as follows:

Clause 2. Stresses to be taken into account.—Secondary stresses have been separated from Deformation stresses and included as an item.

Clause 6. Forces due to curvature of track.—The B. E. S. A. Specification takes no account of the effect of superelevation, which partly or wholly counteracts the centrifugal effect. This has been corrected.

Clause 7. Deformation Stresses.—These have been carefully defined as the bending stresses in members caused by the vertical deflection of the span and the rigidity of the joints. Any stresses within this definition need not be calculated. The reason for this is that even if they exceed the elastic limit the only immediate or ultimate result is that they cannot occur again. A caution is added to take into consideration the effect on the rivets.

Clause 8. Secondary Stresses.—These have been defined and included.

Clause 9. Wind Pressure.—The only alteration is in the factor for including the effect on the lee girders: and the old rules have been followed as more suitable to average Indian conditions.

Clause 10. Braking Forces, etc.—This has been simplified and based on the actual braked axles, and not on total weight of train as in the B. E. S. A. Specification. New girders are to be designed for fully braked trains.

Clause 13. Relief of Stress.—This clause is important and contains a new idea.

Clause 14. Combined Stresses.—In place of the complicated and ambiguous rule of the B. E. S. A., it is proposed to follow the old Bridge Rules and permit a total stress 25 per cent. greater than the normal working stress to cover, dead load, live load, impact, wind, secondary, braking and temperature stresses.

The inclusion of the last three items is new. It is to be noted that "secondary" stresses do not include "deformation" stresses but mean destructive stresses other than axial stresses.

(3) Section (a) III has been omitted entirely, as the Indian Standards of loading have been specified in section (a) II paragraph 4 and it is not considered that the English Standards have any application to Indian conditions.

(4) For section (b) the B. E. S. A. Specification has been adopted with four minor additions.

(5) It is not proposed to alter sections (c), (d) and (e).

(6) As regards section (f) the Committee propose to incorporate a definite rule to allow reduced impact on old bridges at reduced speeds. Various recommendations for reduced impact allowance and also for increased working stresses in conjunction therewith have been proposed in the past. There seems to be point in reducing the impact allowance to say 10 or 20 per cent. and increasing the working stresses by a like amount. This is the same in effect as making no allowance for impact. The following rule is proposed :—

"On old girders where a restriction of 10 miles per hour or less is *rigidly* enforced, no allowance for impact need be made, subject to the responsible authority certifying that the condition of the bridge and of the permanent way warrant this being done. The Transportation Department are to be held responsible that the restriction is rigidly observed. If the restriction is not observed the relaxation of the impact factor cannot be allowed and any permission given for increased loads on the strength of this relaxation must be at once cancelled."

It is recommended that the Restriction Notice Board should shew the minimum time that must be taken between the restriction limits, and further that recording instruments should be used to detect infringements.

For plate girders retained in the road without speed restriction the following addition to section (f) is proposed :—

"Plate girders and rolled joist spans up to and including 40 feet clear span may be retained in use without speed restriction provided the calculated flange and web stresses with full impact allowance do not exceed the specified working stresses by more than 25 per cent. The bearing stresses on the rivets connecting web and flange if calculated by the B. E. S. A. formula with full impact may be taken as 18 tons per square inch for steel and 15 tons per square inch for iron."

SECTION (a) II. LOADS AND STRESSES.

GENERAL.

(1) The following rules apply to fixed span bridges up to 350 feet span between bearings :—

Where bridges of the through or half through type are adopted, they shall be designed with the clearances specified in the Schedule of Maximum and Minimum Dimensions.

Stresses to be taken into account.

(2) For the purpose of computing the maximum stresses in any girder or member of a bridge, each of the following items shall be taken into

account, where applicable, in accordance with the requirement specified herein :—

	See Clause.
1. Dead load	(3)
2. Live load	(4)
3. Impact	(5)
4. Forces due to curvature or eccentricity of Track	(6)
5. Deformation stresses	(7)
6. Secondary stresses	(8)
7. Wind pressure and other forces causing lateral deflection	(9)
8. Longitudinal forces	(10)
9. Temperature effects	(11)
10. Erection stresses	(12)

Dead Load.

(3) Dead load carried by a girder or member shall consist of that portion of the weight of the superstructure, and fixed loads carried thereon, which is supported wholly or in part by the girder or member (including its own weight).

Live Load.

(4) The live load is specified by the Railway Board from time to time. At present there are three standards :—

Standard I. B. G. & M. G.	= Standard B of 1903.
Standard II „ „	= Standard B of 1923 + 25 per cent.
Standard III „ „	= B. B. & C. I. Ry. Standard of 1916.

The live load for Road and Foot Bridges must be according to local requirements. The tables in Appendix I of the B. E. S. A. Specification No. 153, Parts 3, 4 and 5 of 1923 are useful in this connection.

Every design submitted for sanction must bear a certificate showing the Standard of Loading, the Impact Formula and Bridge Rules used in its design.

Impact Effect.

I. RAILWAY BRIDGES.

(5) (a) *New Bridges.*—The impact allowance to be used in calculating the increment of stress that may be expected in the members of any span due to speed is to be taken as equal to the equivalent live load at the position of the train giving the greatest stress in the member multiplied by a factor

$$i = \frac{65}{45n + L}$$

where L = train giving the maximum stress in the member being considered

n = number of tracks supported on two main girders when considering the members of main girders, but equal to unity always for cross girders.

(b) *Existing Girders*.—For the purpose of computing impact on existing bridges the corresponding factor for calculating the increment of live load stress is to be taken as follows :—

- (1) For single and double track spans of 120 feet and over, and for all single track spans when “ n ” equals unity,

$$i\% = \frac{n \ 1050 \ P_1}{(w+p) \ c \ d}$$

where P_1 = resultant max. “ hammer-blow ” in tons of all the out-of-balance weights on the driving wheels of that engine in use which has the greatest vertical unbalanced effect, measured at one revolution per second.

If it is desired to take a value of P_1 less than .6, it must be justified by experimental evidence for the present. For the method of determining P_1 , see Appendix A.

w = equivalent uniformly distributed dead load of the span in tons per foot run.

p = equivalent uniformly distributed live load of the train in tons per foot run.

c = circumference of the loco : driving wheels in feet.

d = deflection of the span in inches under the load “ p ”.

n = 1, for one track and one locomotive.

2, for one tract and two locomotives.

2, for two tracks and for two or more locomotives.

- (2) For single and double track spans below 120 feet when “ n ” = 2,

$$i\% = \frac{n \ 1050 \ P_1}{(w+p) \ c \ d} \left\{ 1 + \frac{\text{span} - 30}{90} \right\}$$

NOTE.—In no case is the Impact allowance to exceed 100 per cent. Impact on hip verticals to be taken the same as for a cross girder.

II. ROAD BRIDGES OVER A RAILWAY.

Provisionally the impact allowance for road bridges over a railway is to be taken as an increment factor to the full live load

$$i = \frac{45}{45 + L}$$

where L = span in feet.

Consideration must be given to the local requirements and the standards of loading laid down by the Local Governments.

III. FOOT-BRIDGES OVER A RAILWAY.

No impact allowance need be made provided the loading Standard is taken at not less than 1 cwt., per sq. foot of deck area.

IV. COMBINED RAIL AND ROAD BRIDGES.

(a) *Main Girders*.—The full impact allowance for train loads only as defined in I is to be made. No additional impact need be allowed for the roadway loads.

(b) *Cross Girders and Floor Members* :—

- (i) For cross girders and floor members carrying the railway, impact is to be allowed as laid down in I.

- (ii) For cross girders and floor members carrying the railway, impact is to be allowed as laid down in II.

Forces due to curvature of track.

(6) Where the track on any bridge is curved the following must be considered:—

- (a) Extra stress in one girder due to the track not being central.
- (b) Extra stress in the outer girder due to overturning effect of centrifugal force on the load.
- (c) Lateral bending of the span due to horizontal centrifugal force transmitted which has the same effect as an increase of wind pressure.

As regards (a), the increase of stress will be proportional to variation of the alignment from the centre of the span.

As regards (b), in practice the track is usually canted on a curve and this has the effect of counteracting the excess weight on the outer girder. In this case the extra load in tons per foot on the other girder

$$= W \frac{H}{D} \left\{ \frac{V^2}{15 R} - \frac{S}{G} \right\}$$

where W = weight of train in tons per foot run—(impact not to be included).

V = speed in miles per hour.

R = radius of curve in feet.

S = superelevation in inches.

G = gauge in inches.

H = height in inches of centre of gravity of train from rail level.

D = distance apart in inches of girders.

As regards (c), the horizontal centrifugal force

$$C = \frac{W V^2}{15 R}$$

where C = centrifugal effect in tons per foot run of span.

In the application of the formula (b) to the case of dead slow speed (*i.e.* $V=0$) the correction for cant indicates a reduction of load on the outer girder and a corresponding increase of load on the inner girder. In general, however, this excess of load on the inner girder can be ignored since at dead slow there is no impact effect.

Deformation Stresses.

(7) A Deformation stress is defined as the bending stress in any member caused by the vertical deflection of the girder combined with rigidity of the joints. No other stresses are included in this definition.

Bridges shall be designed so as to avoid redundant members and to minimise deformation stresses as far as possible. In well designed girders deformation stresses need not be calculated, as they tend to relieve themselves if they are excessive and are not cumulative in their effects. It must be remembered however that deformation stresses are always accompanied by bending moment stresses in the joints and these must be allowed for when considering the group of rivets connecting the member to the gusset.

NOTE.—The above does *not* mean that additional stresses caused by bad design, and such as eccentricity of connections, etc., can be ignored.

Secondary Stresses.

(8) The term secondary stress is here meant to cover bending stresses in the members of trusses additional to axial stresses and deformation stresses as defined in paragraph (7). They arise from eccentricity of connections, the load rolling direct on top booms, cross girders not being connected

at the panel points, wind stresses on the end posts of through girders, etc. In contrast to Deformation stresses these stresses are not self-relieving, so they must be provided for.

In the case of a load rolling on a boom, which therefore acts as a continuous beam, to find the bending stresses in the boom, calculate the bending moment as for a single span of length equal to the panel and take three-fourths of this as being the Bending Moment both at the centre of the member and over the panel points.

A similar method of calculation may be used for rail bearers provided the attachments at their ends are strong enough to ensure their acting as continuous beams.

Wind Pressure.

(9) I. *Allowance per square foot.*—Unloaded bridges up to 300 feet span must ordinarily be designed for a pressure of 50 lbs. per square foot acting on the nett exposed area of the girders.

Loaded bridges shall be designed for pressure of 30 lbs. per square foot on the nett exposed area of the girders *plus* such part of that train as is not shielded by the windward girder. For bridges above 300 feet span and bridges in specially exposed or sheltered situations, the unit pressure must be fixed by the local circumstances.

II. *Area to be considered.*—For unloaded spans the exposed area shall be considered as equivalent to the horizontal projection (or side elevation) of the windward girder multiplied by one of the following factors :—

- 1.0 for plate girders with solid deck.
 - 1.25 for plate girders with open deck.
 - 1.5 for open web girders with solid deck.
 - 1.75 for open web girders with open deck.
- The factor 1.75 shall also be used for trestles.

For loaded spans the nett exposed area shall be computed as the sum of :—

- (a) that portion of the horizontal projection of the windward girder above or below the train area, and
- (b) the train area.

The factor for the case is to be applied to (a).

The area of the train is to be taken as from 2 feet above rail level, to the top of the highest stock using the bridge.

III. *Effects to be considered.*—The effects of wind pressure to be considered are :—

- (a) Lateral bending of the top booms and wind bracing considered as a horizontal girder.
- (b) The same effect on the lower booms.
- (c) An extra vertical load on the leeward main girder due to the additional pressure on the lee rail.
- (d) Secondary stresses in the members transmitting the wind load from the top to the bottom booms, or *vice versa*.

On plate girders up to 60 feet it is not necessary to calculate wind stresses, but lateral bracing should be provided designed for a horizontal moving load of 600 lbs. per foot run.

Longitudinal Forces.

(10) The Breaking Force applied at the rail is to be taken as one-seventh of the load of the braked axles.

The Tractive Force is never greater than, and cannot exist at the same time as the Braking Force; therefore it is not necessary to provide for it separately.

In designing girders for future loadings the Braking Force is to be taken as one quarter of the end shear on the span.

Temperature Effect.

(11) Where any portion of the superstructure is not free to expand or contract under variations of temperature, allowance shall be made for the stresses resulting from this condition, the coefficient of expansion for each degree (Fahrenheit) in variation of temperature above or below the normal being taken at 0.000006. The temperature limits shall be specified by the Engineer.

Erection Stresses.

(12) Where erection stresses, combined with the other permissibly co-existent stresses, would produce a working stress in any member or part of the structure in excess of $33 \frac{1}{3}$ per cent. above the specified working stress, such additional material shall be added to the section, or other provision made, as is necessary to bring the working stress within that limit.

Relief of Stresses.

(13) Proposals may be put forward to take advantage of any relief of stress afforded by adjacent parts when determining the maximum stress in any member, both in new and old girders. A note must be incorporated in the drawing and in the stress sheet showing exactly what relief has been claimed and how it is arrived at. Furthermore the covering letter submitting the design for sanction must draw special attention to this matter and justify the proposal.

In every such case it is necessary to consider whether the relief claimed will be given by the adjacent member permanently or is liable to vanish owing to any change in the said adjacent member. As an example, relief of stress in cross girders may be claimed due to the partial fixation of their ends by the vertical posts to which they are attached. But such partial fixation induces high bending stresses in the post, and if these are so great as to cause permanent bending of the post, part or all of the end fixation of the cross girder will be lost. Therefore if such relief be claimed the bending stresses in the posts must be estimated and allowed for.

Combined Stresses.

(14) The sum of the stresses caused by Dead Load, Live Load, Impact, and Curvature of Track (if any), subject to any duly sanctioned relief afforded by adjacent parts, must not exceed the normal limits of stress specified. The sum of the above stresses together with secondary stresses and those due to wind, Longitudinal Forces and Temperature should not exceed the normal stress by more than 25 per cent.

Anchorage.

(15) Anchorage shall be provided against longitudinal and lateral movement, and also to the extent of 50 per cent. in excess of any possible overturning moment of the span as a whole, or of the knuckles, due to wind or longitudinal forces.

(b) SPECIAL RULES FOR STEEL AND IRON BRIDGES.

Clauses 17 to 26 of Part 3 and Clauses 1 to 34 of Part 4 included in the B. E. S. A. Specification No. 153 of 1923 are to be taken as the special

rules for Steel and Iron Bridges, with the following alterations and additions :—

Part 3. (17) *Add at end—*

“The rivets in the end connections must be proportioned to the sum of the maximum tensile and compressive forces in the member.”

(22) *Add—*

“For shear stresses in wrought iron take 80 per cent. of the corresponding stress for steel.”

Part 4. (4) *Add—*

“As an alternative Waddell's method of determining the nett section of tension members may be used: *vide* Technical Paper No. 211-Third Report of I. R. B. C.”

(21) *Add—*

“In special cases it may be necessary to depart from the limits laid down in this clause, but each such case requires special sanction.”

BRIDGE RULES, 1925.

APPENDIX A.

Method of determining the resultant out-of-balance “hammer blow” P_1 of any locomotive.

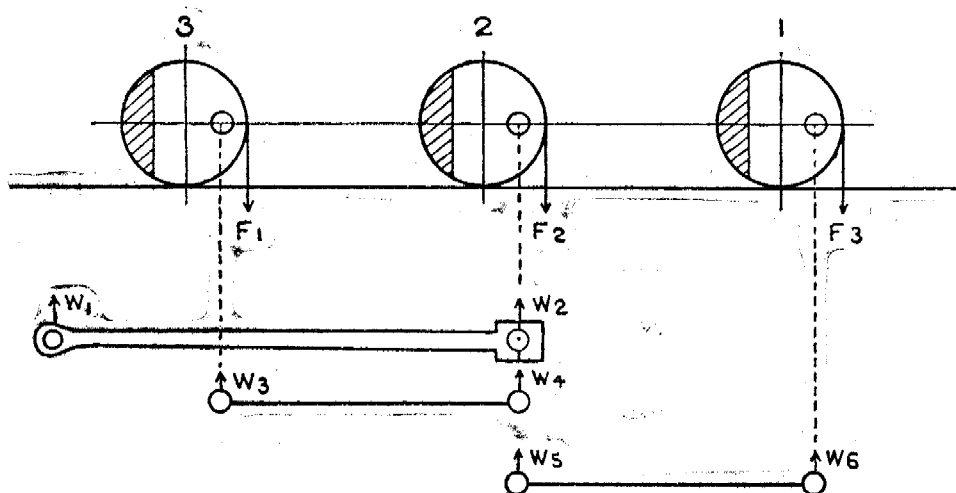
Formula.

$$P_1 = 0.000065 M r.$$

$M r$ is the sum of the nett out-of-balance of all the wheels on one side of an engine with all the motion attached, expressed as a moment in inch lbs.

Shop method of determining nett out-of-balance $M r$.

A simple means of arriving at this figure is given in the diagram below, the forces F_1, F_2, F_3 etc., being measured with a spring balance or otherwise. The axles should be mounted on knife edges for making the tests.



Example outside cylinder engine, coupling rods on same crank pin as connecting rod.

F_1, F_2, F_3 = forces at tyre required to balance counterweight, crank pins, etc.

d = diameter of wheel over tyres in inches.

$\left. \begin{matrix} W_1 & W_2 \\ W_3 & W_4 \\ W_5 & W_6 \end{matrix} \right\}$ = weights of each end of connecting and coupling rods.

r = crank pin radius.

$$\text{Out-of-balance in 3} = F_1 \frac{d}{2} - W_3 r$$

$$\text{,, ,, ,, 2} = F_2 \frac{d}{2} - (W_2 + W_4 + W_5) r$$

$$\text{,, ,, ,, 1} = F_3 \frac{d}{2} - W_6 r.$$

$$\text{Total out-of-balance } M r = (F_1 + F_2 + F_3) \frac{d}{2} - (W_2 + W_3 + W_4 + W_5 + W_6) r$$



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X

Note on a Proposed Revision of the Bridge Rules.

It is proposed, if opinions are favourable, to revise the Chapter in the Rules for Opening a Railway, which deals with the inspection of Railway Bridges, as indicated in the following pages. The new rules would apply for the present to 5'-6" gauge railways only. But it is hoped the rules for other gauges will be settled in the near future.

2. The principal alterations are:—

- (1) The allowance for Impact.
- (2) The method of treating Deformation Stresses.
- (3) The Standards of Loading.

3. *Impact.*—The Committee, by adopting the principles which were suggested in the preliminary proceedings, have been able to arrive at a rational impact allowance considerably below the Pencoyd and B. E. S. A. Formulæ.

It was assumed that the track over the bridge is in reasonably good order, and that the rolling stock and locomotives are properly maintained in ordinary good running order. In short, the conditions under which the experiment, on which the results are based, were made were ordinary running conditions. There was no need to go into the question of engines with a series of flats round the driving wheels, nor stock without springs, nor broken-ended rails in the track. It was also assumed that locomotives are reasonably well balanced in accordance with modern practice, and the limits of good balancing are described in the report. It was also assumed that trains run at normal working speeds, and that heavy goods trains do not run at 70 m.p.h. It was further assumed that in accordance with reason the impact produced by loads below the standard for the bridge need not be taken into account. The Committee follow the precedent set by the American Committee in making use of deflection tests only for purposes of deciding the amount of impact. Tests of individual members with stress recorders are always unreliable being affected by deformation stresses. By adopting these principles the Committee were able to eliminate a large number of otherwise inexplicable experimental results which have been a stumbling block to previous investigators.

Guided by these principles and by the work of Prof. Inglis, they have arrived at a practical working formula for impact, by the use of which we may confidently increase our loading without the need for a factor of ignorance. The proposed allowance for impact agrees with my experience of what is necessary.

4. The only matter, on which we remain in ignorance, is the question of variation of impact in the different members. Here the Committee have fallen back on the old method of calculating impact in terms of the loaded length and in this way provide for the assumption that web members required higher impact allowance than chords. I think that this assumption is based on inconclusive evidence.

5. The Committee have adopted the view, with which I agree, that Deformation stresses may be ignored. Their reasons are given in the report. They retain the need for providing for these stresses in the joints but leave it to Engineers to decide what provision may be required.

6. The standards of loading now proposed are:—

Standard H. M.—This standard is recommended for use on all important railways where a heavy mineral traffic is anticipated. It provides for the heaviest locomotives with 30-ton axle loads running on 120 lb. rails. The train load behind the engine is 2·5 tons per foot.

It is anticipated that the high cost of transporting coal in this country owing to the exceptionally long lead will compel the introduction of the recommended running dimensions of 1923 at an early date. With these dimensions it will be possible to build large locomotives and wagons and in this way bring down the cost of moving coal which is the most urgent need of the country. Cheap coal will not only help industry but, what is still more important in an agricultural country, it will reduce the cost of transport. A diagram of the train is attached with the equivalent loading.

Only the spans up to 12 feet and from 100 feet to 200 feet of the old Standard III girders will carry this loading.

Standard I of 1925 may be used on all important railways where heavy mineral traffic is not anticipated. In view, however, of the small extra cost of Standard H. M. and the fact that girders have twice the life of rails, it is considered that Standard I girders should only be used where Standard H. M. is unlikely ever to be needed.

Standard I provides for $22\frac{1}{2}$ -ton axle loads on engine and wagons and the train load behind the engine is 2.5 tons per foot run.

The rails corresponding will be 90 lb. per yard. A diagram of the train is attached and the equivalent loading. The old Standard III girders will carry this loading with a large margin, and up to 15 ft. span B+25 per cent. will carry it.

Standard II of 1925.—This is for Branch line using 75 lb. rails and worked by locomotives of the 2-8-0 type with 17-ton axle load and drawing trains of the existing 32-ton wagons. The old Standard B girders are strong enough for this Standard with a small overload in some cases, which might be ignored for goods train speeds. The new wagons with $22\frac{1}{2}$ -ton axle load will be worked on the branches if required subject to a restricted load and speed.

7. The method of dealing with centrifugal force recommended by the Sub-Committee has not been adopted. It is considered that the B. E. S. A. rule is sufficiently detailed and that rule has been retained, with a note to draw attention to the effect of cant.

(Sd.) L. E. HOPKINS,
Director, Civil Engineering, Railway Board.



XI

**Proposed Bridge Rules for 5'-6" Gauge Railways
with existing Bridge Rules, 1923, printed
on opposite pages for comparison.**

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EXISTING BRIDGE RULES, 1923.

Rules for the Inspection of Railway Bridges.**(a) GENERAL RULES APPLICABLE TO ALL NEW BRIDGES.****1. Duties of Government Inspector and Railway Administrations—**

1. For calculations dealing with the strength of a railway bridge and for the determination of the loads for which such a bridge may be used, the following rules are to be adopted so far as they are applicable to the case; in the event of any particular structure failing to come up to the standard here specified, a Senior Government Inspector of Railways should make a special recommendation as to whether the structure may be used and, if so, under what conditions.

2. Except under the special sanction of the Senior Government Inspector of Railways for the circle concerned, no load may be imposed on any railway bridge which would cause in any member thereof stresses greater than those specified in Rules 29, 31, 36 and 42-43.

3. Except under similar sanction, no railway bridge may be erected or re-opened for traffic after strengthening unless it is able to carry, without exceeding the stresses specified in these Rules, the loads specified in Rules 6-24.

4. In circumstances where a first class standard is not required (as in the case of a temporary bridge or of a railway worked at low speed or with small axle-loads), the Inspector may recommend a departure from these rules on conditions to be specified by him, provided the calculations forwarded with his report show that opening on such conditions will not be attended with danger to the public or the running staff.

5. Any Railway Administration which desires to use bridges, track, or rolling-stock differing from the standards prescribed in these rules, may apply for sanction to do so, and the proposals will be considered on their merits. The application must be accompanied by—

(a) Such diagrams, etc., as may be necessary to give full particulars of the axle-loads, wheel-spacing, length over buffers and other principal dimensions of the rolling-stock for which sanction is required.

(b) A certificate signed (or countersigned) by the Principal Technical Adviser to the Administration, showing that the use of such rolling-stock (over such sections of the line, and under such conditions as he may see fit to specify) will not involve danger to the traffic or abnormal injury to existing structures or track.

(c) Such calculations, stress sheets, etc., as may be necessary to show how the conclusion is arrived at, the external forces on which the stress calculations are based, the stresses which will be produced in the various bridges over which the proposed rolling-stock will run, and the effects which the said rolling-stock will have on various structures or tracks as compared with those caused by rolling-stock already in use, or allowed by existing Government orders: the calculations, stress sheets, etc., must show clearly what allowance has been made for any *secondary stresses* which may be produced in addition to the primary stresses caused by the external forces.

(d) An approximate estimate of the cost of such improvements, in existing structures or track, as the use of the proposed rolling-stock is likely to render necessary on the railway concerned, whether immediately or in the near future.

It is desired that the calculations, etc., submitted in support of such applications should, so far as practicable, follow the methods prescribed in these Rules, whether any other alternative method of calculation is adopted also or not.

The calculations, etc., must be scrutinised by the Inspector, and his recommendations thereon submitted to the Railway Board for orders; the bridges, tracks, structures of rolling-stock may not be ordered or brought into use without the sanction of the Railway Board.

PROPOSED BRIDGE RULES.

Rules for the Inspection of Railway Bridges.

(a) GENERAL RULES APPLICABLE TO ALL NEW BRIDGES.

1. Duties of Government Inspector and Railway Administrations—

1. For calculations dealing with the strength of a railway bridge and for the determination of the loads for which such a bridge may be used, the following rules are to be adopted so far as they are applicable to the case; in the event of any particular structure failing to come up to the standards here specified, a Senior Government Inspector of Railways should make a special recommendation as to whether the structure may be used and, if so, under what conditions.

2. Except under the special sanction of the Senior Government Inspector of Railways for the circle concerned, no load may be imposed on any railway bridge which would cause in any member thereof stresses greater than those specified in these Rules.

3. Except under similar sanction, no railway bridge may be erected or re-opened for traffic after strengthening unless it is able to carry, without exceeding the stresses specified in these Rules, the loads specified in Rules G-38.

4. In circumstances where a first class standard is not required (as in the case of a temporary bridge or of a railway worked at low speed or with small axle-loads), the Inspector may recommend a departure from these rules on conditions to be specified by him, provided the calculations forwarded with his report show that opening on such conditions will not be attended with danger to the public or the running staff.

5. Any Railway Administration which desires to use bridges, track, or rolling-stock differing from the standards prescribed in these rules, may apply for sanction to do so, and the proposals will be considered on their merits. The application must be accompanied by—

- (a) Such diagrams, etc., as may be necessary to give full particulars of the axle-loads, wheel-spacing, length over buffers and other principal dimensions of the rolling-stock for which sanction is required.
- (b) A certificate signed (or countersigned) by the Principal Technical Adviser to the Administration, showing that the use of such rolling-stock (over such sections of the line, and under such conditions as he may see fit to specify) will not involve danger to the traffic or abnormal injury to existing structures or track.
- (c) Such calculations, stress sheets, etc., as may be necessary to show how the conclusion is arrived at, the external forces on which the stress calculations are based, the stresses which will be produced in the various bridges over which the proposed rolling-stock will run, and the effects which the said rolling-stock will have on various structures or tracks as compared with those caused by rolling-stock already in use, or allowed by existing Government orders: the calculations, stress sheets, etc., must show clearly what allowance has been made for any *secondary stresses* which may be produced in addition to the primary stresses caused by the external forces.
- (d) An approximate estimate of the cost of such improvements, in existing structures or track, as the use of the proposed rolling-stock is likely to render necessary on the railway concerned, whether immediately or in the near future.

It is desired that the calculations, etc., submitted in support of such applications should, so far as practicable, follow the methods prescribed in these Rules whether any other alternative method of calculation is adopted also or not.

The calculations, etc., must be scrutinised by the Inspector, and his recommendations thereon submitted to the Railway Board for orders; the bridges, tracks, structures of rolling-stock may not be ordered or brought into use without the sanction of the Railway Board.

EXISTING BRIDGE RULES, 1923.

II. Loads or External Forces for which stresses must be calculated—

6. A railway bridge must be designed to sustain the greatest stresses produced in any part of it by the "total working load," *i.e.*, by any combination of the following external forces which can occur simultaneously :

- (a) Fixed load.
- (b) Moving load.
- (c) Wind and other forces causing lateral deflection and transverse racking.
- (d) Traction and braking forces.
- (e) Forces due to horizontal curvature of track.
- (f) Forces due to changes of temperature.

NOTE.—Stresses produced by external forces comprise—

- (1) "Primary", "direct" or "axial" stresses and, usually,
- (2) "Secondary" or "Deformation" stresses.

"Primary stresses" are the stresses which would be produced if certain broad assumptions, which are usually introduced to simplify calculations, were true.

A "Secondary stress" is merely a correction to allow for the approximate nature of the assumptions on which the calculations of a primary stress are based.

Calculations hitherto submitted to the Railway Board have usually referred only to primary stresses based on assumptions which are not fulfilled in actual design or construction.

In future it will be necessary for secondary stresses, where such exist, to be calculated or otherwise allowed for.

Nevertheless, it may not be necessary to make many calculations for secondary stresses in well-designed bridges of small or moderate span : for it is probable that secondary stresses of average moderate values (perhaps 25 per cent. of the primary stresses) are allowed for in a margin between the "elastic limit" and the "permissible stresses" specified in Rules 29, 31, 36 and 42-43.

7. *Fixed load* comprises the weight of those parts of a bridge (including earth cushion, roadway, flooring, ballast, permanent-way, etc.,) which from their position cause stresses in the bridge or in any part of the bridge which is under consideration.

8. *Moving load* is to be taken as the vertical train load specified in Rule 15 moving in its normal direction on each line of rails, together with the additional load specified in Rule 16 if there be a road or footway that can be occupied at the same time as the railway track or tracks.

III. Standards of Train-Load and other External Forces.

15. *Train-Load*.—The Train-Load shall be preferably a multiple of one of the systems of unit loadings set out in the Appendix to Part 3 (Loads and Stresses) of the British Standard Specification for Girder Bridges.

Suitable multiples are as follows :—

For important 5' 6" gauge railways,—B. S. 4' 8½" gauge unit loading	× 22½
For less " " " " " " " " " "	× 17
For light " " " " " " " " " "	× 12
For important 3' 3¾" " " " " " " " " "	× 17 or 12
For light " " " " " " " " " "	× 8
For " 2' 6" " " " " " " " "	× 12, 8 or 5
For " 2' 0" " " " " " " " "	× 8, 5 or 3

Other multiples and other loadings may be sanctioned by the Railway Board if recommended by the Railway Administration concerned.

16. *Road and Footway Loads*.—For a road or foot-bridge over a railway or for a combined road and railway bridge, the moving load on the road or footway shall be taken as the heaviest crowd or collection of elephants, cattle, cannon, traction-engines, road-rollers, motor lorries or other vehicles (combined with a crowd) which is likely to be allowed on the bridge. (Now printed as appendix B (1) of proposed Bridge Rules.)

17. Particulars of the weights and dimensions of various loads likely to traverse roads in India are given in Table IV.

PROPOSED BRIDGE RULES.

II. *Loads and stresses.*

GENERAL.

6. The following rules apply to fixed span bridges up to 350 feet span between centres of bearings.

Where bridges of the through or half through type are adopted, they must be designed with the clearances specified in the Schedule of Maximum and Minimum Dimensions used on Indian Railways.

STRESSES TO BE TAKEN INTO ACCOUNT.

7. For the purpose of computing the maximum stresses in any girder or member of a bridge, each of the following items shall be taken into account, where applicable, in accordance with the requirement specified herein:—

- (a) Dead load.
- (b) Live load.
- (c) Impact.
- (d) Forces due to curvature or eccentricity of Track.
- (e) Deformation stresses.
- (f) Secondary stresses.
- (g) Wind pressure and other forces causing lateral deflection.
- (h) Longitudinal forces.
- (i) Temperature effects.
- (j) Erection stresses.

8. DEAD LOAD carried by a girder or member shall consist of that portion of the weight of the superstructure, and the fixed loads carried thereon, which is supported wholly or in part by the girder or member (including its own weight).

9. LIVE LOAD is specified by the Railway Board from time to time. For the present there will be three standards:—

Standard H. M., recommended for all lines carrying heavy mineral traffic.

Standard I, for use with 90 lbs. track and $22\frac{1}{2}$ ton axle-loads.

Standard II, for use with 75 lbs. track and 17 ton axle-loads.

NOTE:—Diagrams of the trains and equivalent loads will be found in Appendix B (2).

10. The live load for Road and Foot Bridges must be according to local requirements. The tables in Appendix I of the B. E. S. A. Specification No. 153, Parts 3, 4 and 5 of 1923, are useful in this connection.

11. Particulars of the weights and dimensions of various loads likely to traverse roads in India are given in Appendix B (1).

EXISTING BRIDGE RULES, 1923.

15 (contd.) Every drawing submitted to the Railway Board must bear a certificate in the following form:—“*Train-Load*:—This bridge has been designed under the Bridge Rules of 1923 (or 1903) to carry a train-load which produces { Bending Moments per cent. above or below } the scale of loading known as the 5' 6" (or metre) gauge Standard B of 1903.”

NOTE.—(a) The two standards “B of 1903” are laid down in Tables I, II and III appended to these rules. (Not printed.)

(b) No 2' 6" gauge locomotive or other rolling-stock will ordinarily be sanctioned by the Railway Board for a railway in the plains if it will produce greater Bending Moments, Shearing Forces or Cross Girder Reactions than the “Metre Gauge Standard B of 1903.”

18. Road and footway loads should be compared with the unit load for road bridges printed in the Appendix to Part 3, (Loads and Stresses), of the British Standard Specification. Every drawing submitted to the Railway Board should bear a certificate in the following form:—

“*Road (and/or Footway) Load*.—This bridge has been designed under the Bridge Rules of 1923 (or 1903) to carry a road (and/or foot-

way) load which produces { Bending Moments } approxi-
Shearing Forces
Cross Girder Reactions }

mately equal to those produced by { } times the British

Standard Unit Loading for Road Bridges.”

13. All the loads [A, B, C, D, E, F] except B, Moving Load, are to be considered as Static Loads. To convert the Moving Load to its equivalent static load an appropriate increment must be allowed for Impact.

14. *Impact* may be defined, for the purposes of these rules, as “the difference between the vertical effect on a bridge of a load moving over the bridge and the effect of the same load at rest on the bridge.” The increment for Impact to be allowed in various cases is prescribed in rules 26—28, 30, 33—34 and 41.

26. *Impact*.—The Impact Factor $\frac{120}{90 + \frac{n+1}{2}L}$ which is described in sub-

clause 5 (a) of Part 3 of the British Standard Specification shall apply to train-loads on spans up to 50 feet only.

27. For spans over 50 feet, the increment for Impact (see rules 13 and 14) is to be calculated, for steel and iron bridges under railways of all

gauges, by multiplying the train-load by the factor $\frac{300}{300+L}$

PROPOSED BRIDGE RULES.

12. Every design submitted to the Railway Board for sanction must bear a certificate showing the Standard of Loading, the Impact Formula and Bridge Rules used in its design.



13. *Impact effect.*—The impact allowances specified below for each type and kind of bridge shall be used:—

(1) RAILWAY BRIDGES.

(a) *New Bridges.*—The impact allowance to be used in calculating the increment of stress that may be expected in the members of any span due to speed is to be taken as equal to the equivalent live load at the position of the train giving the greatest stress in the member multiplied by a factor

$$i = \frac{65}{45n + L}$$

subject to a maximum value of 1.00 for spans up to 20 feet.

where L = loaded length of the span at the position of the train giving the maximum stress in the member being considered.

n = number of tracks supported on two main girders when considering the members of main girders, but equal to unity always for cross girders.

EXISTING BRIDGE RULES, 1923.



28. For moving loads on a road or foot bridge over a railway, or on the road-way on foot-ways of a combined road and railway bridge, the Impact Factors for spans up to, and over, 50 feet shall be respectively

$$\frac{1}{2} \text{ of } \frac{120}{90 + \frac{n+1}{2} L} \text{ and } \frac{1}{2} \text{ of } \frac{300}{300 + L}.$$

NOTE :—The Railway Board are willing to consider recommendations for smaller Impact Factors than those prescribed in rules 26-28 in any particular case, such as that of a well balanced electric or steam locomotive, provided that proof is produced that the factors prescribed in rules 26-28 are excessive for the particular case in question.

PROPOSED BRIDGE RULES.

(b) *Existing Girders*.—For the purpose of computing impact on existing bridges the corresponding factor for calculating the increment of live load stress is to be taken as follows:—

- (i) For single and double track spans of 120 feet and over, and for all single track spans when “n” equals unity,

$$i\% = \frac{n \cdot 1050 \cdot P_1}{(w+p) \cdot c \cdot d}$$

where P_1 = resultant max. “hammer-blow” in tons of all the out-of-balance weights on the driving wheels of that engine in use which has the greatest vertical unbalanced effect measured at one revolution per second. If it is desired to take a value of P_1 less than ‘6’, it must be justified by experimental evidence for the present. For the method of determining P_1 see Appendix A.

w = equivalent uniformly distributed dead load of the span in tons per foot run.

p = equivalent uniformly distributed live load of the train in tons per foot run.

c = circumference of the loco. driving wheels in feet.

d = deflection of the span in inches under the load “p”.

n = 1, for one track and one locomotive.

2, for one track and two locomotives.

2 for two tracks and for two or more locomotives.

- (ii) For single and double track spans below 120 feet, when “n”=2,

$$i\% = \frac{n \cdot 1050 \cdot P_1}{(w+p) \cdot c \cdot d} \left\{ 1 + \frac{\text{span}-30}{90} \right\}$$

NOTE.—In no case is the impact allowance to exceed 100 %. Impact on hip verticals is to be taken the same as for a cross girder.

(2) ROAD BRIDGES OVER A RAILWAY.

Provisionally the impact allowance for road bridges over a railway is to be taken as an increment factor to the full live load.

$$i = \frac{45}{45+L}$$

Where L = span in feet.

Consideration must be given to the local requirements and the standards of loading laid down by the Local Governments.

(3) FOOT-BRIDGES OVER A RAILWAY.

No impact allowance need be made provided the loading standard is taken at not less than 1 cwt. per square foot of deck area.

(4) COMBINED RAIL AND ROAD BRIDGES.

(a) *Main Girders*.—The full impact allowance for train loads only as defined in (1) is to be made. No additional impact need be allowed for the roadway loads.

(b) *Cross Girders and Floor Members*.—

- (i) For cross girders and floor members carrying the railway, impact is to be allowed as laid down in (1).

- (ii) For cross girders and floor members carrying the roadway, impact is to be allowed as laid down in (2).

EXISTING BRIDGE RULES, 1923.

12. *Forces due to Horizontal Curvature of track and Forces due to changes of Temperature* are dealt with in rules 23 and 24.

23. *Loads due to Curvature of the track* are (1) a horizontal force, (acting at the rails of each track in a direction at right angles to the track), which should be computed as that fraction of the train-load given by the formula $\frac{V^2}{15 R}$ in which V is the velocity of the train in miles per hour and R is the radius of the curve in feet, and (2) an increment to the train-load on the outer rail due to the tendency of the train to overturn when moving.

NOTE.—See notes (ii) below Tables I and II regarding a further increment to the train-load on the outer rail to be allowed whether the train is moving or at rest.

Note to existing Rule 6.

NOTE.—Stresses produced by external forces comprise—

- (1) "Primary", "direct" or "axial" stresses and, usually,
- (2) "Secondary" or "Deformation" stresses.

"Primary stresses" are the stresses which would be produced if certain broad assumptions, which are usually introduced to simplify calculations, were true.

A "Secondary stress" is merely a correction to allow for the approximate nature of the assumptions on which the calculations of a primary stress are based.

Calculations hitherto submitted to the Railway Board have usually referred only to primary stresses based on assumptions which are not fulfilled in actual design or construction.

In future it will be necessary for secondary stresses, where such exist, to be calculated or otherwise allowed for.

Nevertheless, it may not be necessary to make many calculations for secondary stresses in well-designed bridges of small or moderate span: for it is probable that secondary stresses of average moderate values (perhaps 25 per cent. of the primary stresses) are allowed for in a margin between the "elastic limit" and the "permissible stresses" specified in Rules 29, 31, 36 and 42—43.

PROPOSED BRIDGE RULES.

14. *Forces due to curvature of track.*—Where the track on any bridge is curved the following must be considered :—

- (a) Extra stress in one girder due to the track not being central.
- (b) The centrifugal force.

15. As regards (a), the increase of stress may be calculated in each case.

16. As regards (b), horizontal centrifugal force, which may be assumed to act at a height of 6 feet above rail level, is

$$C = \frac{W V^2}{15 R}$$

Where C is the horizontal effect in tons per foot run of span

W = equivalent distributed live load in tons per foot run

V = speed of train in miles per hour

R = radius of curve in feet.

The excess load on the inner girder due to the cant may be ignored, as this excess only occurs at low speeds when there is no impact.

NOTE.—With the maximum curvature allowed for unrestricted speed and with the normal allowance for cant, the horizontal centrifugal force, which must be provided for at Rail Level, is about $\frac{1}{12}$ th of W, and the load on the outer rail about $\frac{1}{20}$ W in excess of the load on the inner rail.

18. *Deformation Stresses.*—A Deformation stress is defined as the bending stress in any member caused by the vertical deflection of the girder combined with rigidity of the joints. No other stresses are included in this definition.

19. Bridges shall be designed so as to avoid redundant members and to minimise deformation stresses as far as possible. In well designed girders deformation stresses need not be calculated, as they tend to relieve themselves if they are excessive and are not cumulative in their effects. It must be remembered however that deformation stresses are always accompanied by bending moment stresses in the joints and these must be allowed for when considering the group of rivets connecting the member to the gusset.

NOTE.—The above does *not* mean that additional stresses caused by bad design, such as eccentricity of connections, etc., can be ignored.

20. *Secondary Stresses.*—The term secondary stress is here meant to cover bending stresses in the members of trusses additional to axial stresses and Deformation stresses as defined in Rule 18. They arise from eccentricity of connections the load rolling direct on top booms, cross girders not being connected at the panel points, wind stresses on the end posts of through girders, etc. In contrast to Deformation stresses these stresses are not self-relieving, so they must be provided for.

21. In the case of a load rolling on a boom, which therefore acts as a continuous beam, to find the bending stresses in the boom, calculate the bending moment as for a single span of length equal to the panel and take three-fourths of this as being the Bending Moment both at the centre of the member and over the panel points.

22. A similar method of calculation may be used for rail bearers provided the attachments at their ends are strong enough to ensure their acting as continuous beams.

EXISTING BRIDGE RULES, 1923.

9. *Wind load* is to be taken as a horizontal force calculated in accordance with rules 19 and 20.

10. The other forces, included in C (rule 6), causing *lateral deflection* and *transverse racking* are dealt with in rule 21.

19. *Wind Pressure on Unloaded Bridges*.—Unloaded Bridges up to 300 feet span must be capable of resisting a wind pressure of 50 lbs. per square foot. Above this length of span a special determination for wind pressure per square foot must be made, depending on the local conditions.

20. *Wind Pressure on Loaded Bridges*.—In the case of a loaded bridge the intensity of wind pressure may be assumed to be not greater than 30 lbs. per square foot.

The additional pressure on the leeward rail is to be taken into consideration.

21. *The forces referred to in Rule 10*.—Occur only when a train is travelling over a bridge; they are mathematically indeterminate but they must be provided for by rigid lateral and transverse bracing.



PROPOSED BRIDGE RULES.

23. Wind pressure must be allowed for in accordance with the rules given below:—

(1) Allowance per square foot.

24. *Unloaded Bridges* up to 300 feet span must ordinarily be designed for a pressure of 50 lbs. per square foot acting on the nett exposed area of the girders.

25. *Loaded Bridges* shall be designed for a pressure of 30 lbs. per square foot on the nett exposed area of the girders *plus* such part of the train as is not shielded by the windward girder. For bridges above 300 feet span and bridges in specially exposed or sheltered situations, the unit pressure must be fixed by the local circumstances.

(2) Area to be considered.

26. *For unloaded spans* the exposed area shall be considered as equivalent to the horizontal projection (or side elevation) of the windward girder multiplied by one of the following factors:—

1.0 for plate girders with solid deck.

1.25 for plate girders with open deck.

1.5 for open web girders with solid deck.

1.75 for open web girders with open deck.

The factor 1.75 shall also be used for trestles.

27. *For loaded spans* the nett exposed area shall be computed as the sum of:—

(a) that portion of the horizontal projection of the windward girder above or below the train area, and

(b) the train area.

The factor for the case is to be applied to (a).

The area of the train is to be taken as from 2 feet above rail level, to the top of the highest stock using the bridge.

(3) Effects to be considered.

28. The effects of wind pressure to be considered are:—

(a) Lateral bending of the top booms and wind bracing considered as a horizontal girder.

(b) The same effect on the lower booms.

(c) An extra vertical load on the leeward main girder due to the additional pressure on the lee rail.

(d) Secondary stresses in the members transmitting the wind load from the top to the bottom booms, or *vice versa*.

29. On plate girders up to 60 feet it is not necessary to calculate wind stresses, but lateral bracing should be provided designed for a horizontal moving load of 600 lbs. per foot run.

EXISTING BRIDGE RULES, 1923.

11. *Traction and Braking Force* are the *longitudinal forces* caused by the starting and stopping of a train; see rule 22.

22. *Traction and Brake Loads* are to be taken as horizontal forces acting at the rails of each track in the direction of the moving train and are to be computed as directed in clause 9 of part 3 of the British Standard Specification for Girder Bridges.

12. *Forces due to Horizontal Curvature of track and Forces due to changes to Temperature* are dealt with in Rules 23 and 24.

24. *Stresses due to changes of Temperature* are to be computed for a range of temperature of 100 degrees Fahrenheit or for such range as the circumstances of the case render necessary. The co-efficients of expansion per degree Fahrenheit may be taken as .000006 both for steel and for concrete.



PROPOSED BRIDGE RULES.

30. *Longitudinal Forces.*—The Braking Force applied at the rail is to be taken as one-seventh of the load of the braked axles.

31. The Tractive Force is never greater than, and cannot exist at the same time as the Braking Force; therefore it is not necessary to provide for it separately.

32. In designing girders for future loadings the Braking Forces is to be taken as one quarter of the end shear on the span.

33. *Temperature Effect.*—Where any portion of the superstructure is not free to expand or contract under variations of temperature, allowance shall be made for the stresses resulting from this condition, the co-efficient of expansion for each degree (Fahrenheit) in variation of temperature above or below the normal being taken at 0.000006. The temperature limits shall be specified by the Engineer.

34. *Erection Stresses.*—Where erection stresses, combined with the other permissibly co-existent stresses, would produce a working stress in any member or part of the structure in excess of $33\frac{1}{2}$ per cent. above the specified working stress, such additional material shall be added to the section, or other provision made, as is necessary to bring the working stress within that limit.

35. *Relief of Stresses.*—Proposals may be put forward to take advantage of any relief of stress afforded by adjacent parts when determining the maximum stress in any member, both in new and old girders. A note must be incorporated in the drawing and in the stress sheet showing exactly what relief has been claimed and how it is arrived at. Furthermore the covering letter submitting the design for sanction must draw special attention to this matter and justify the proposal.

36. In every such case it is necessary to consider whether the relief claimed will be given by the adjacent member permanently or is liable to vanish owing to any change in the said adjacent member. As an example, relief of stress in cross girders may be claimed due to the partial fixation of their ends by the vertical posts to which they are attached. But such partial fixation induces high bending stresses in the post, and if these are so great as to cause permanent bending of the post, part or all of the end fixation of the cross girder will be lost. Therefore if such relief be claimed the bending stresses in the posts must be estimated and allowed for.

37. *Combined stresses.*—The sum of the stresses caused by Dead Load, Live Load, Impact and Curvature of Track (if any), subject to any duly sanctioned relief afforded by adjacent parts, must not exceed the normal limits of stress specified. The sum of the above stresses together with secondary stresses and those due to wind, Longitudinal Forces and Temperature should not exceed the normal stress by more than 25 per cent.

38. *Anchorage.*—Anchorage shall be provided against longitudinal and lateral movement, and also to the extent of 50 per cent. in excess of any possible overturning moment of the span as a whole, or of the knuckles, due to wind or longitudinal forces.

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(b) SPECIAL RULES FOR STEEL AND IRON BRIDGES—

25. All the clauses of the British Standard Specification for Girder Bridges which are applicable to Indian conditions and do not conflict with these rules shall be treated as a part of these rules and shall be complied with in the design and inspection of steel and iron bridges.

NOTE.—The British Standard Specification leaves various points to be settled at the discretion of “the Engineer (or Purchaser)”: in accordance with the spirit of rules 1--5, above, and 45--48, below, the Railway Board reserve the right to over-rule “the Engineer (or Purchaser)” on such points.

29. *Maximum permissible stresses*.—Intensities of stress (*alias* “Working Stresses” or “Unit Stresses”) due to the “total working load” (see rules 6 and 13) shall not exceed the limits specified in clauses 13--26 of Part 3 of the British Standard Specification for Girder Bridges.



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(b) SPECIAL RULES FOR STEEL AND IRON BRIDGES—

(Reproduced from Parts 3 and 4 of B. E. S. A. Specification No. 153 of 1923.)

39. The following rules shall be complied with in the design construction and inspection of steel and iron bridges:—

40. *Alternating Stresses*.—Members subject to alternating stresses shall be proportioned for tension and compression separately, half the smaller gross area being added to the larger gross area to give the total section necessary. They shall be designed as rigid members to provide for the compression stress. In the case of wind bracing, the members shall be designed as struts to resist the greater stress only. The rivets in the end connections must be proportioned to the sum of the maximum tensile and compressive forces in the member.

41. *Working Stresses in Structural Steel*.—Except as hereinbefore modified, structures shall be so designed that the calculated working stresses in structural steel of the "A" quality specified in Clause I, Part I, of the B. S. Specification No. 153 of 1922 shall not exceed the following:—

For parts in tension:—

On the nett section for avial stress, 8 tons (17,920 lb.) per square inch.

For parts in compression:—

On the gross section of the compression flanges of plate girders and I beams with outside edges stiffened with angles or channels,

$8 \left(1 - 0.0075 \frac{l}{b}\right)$ tons per square inch.

Do. . . . do. . . . with unstiffened edges, $8 \left(1 - 0.01 \frac{l}{b}\right)$ tons per square inch.

where l = the greatest unsupported length, as defined in Rule 10, and b = the breadth of the flange, provided that the gross area of the compression flange, shall be not less than the gross area of the tension flange.

On the gross section of compression members of truss and lattice girders with rivetted connections for axial stress, $8 \left(1 - 0.0033 \frac{l}{r}\right)$ tons per square inch. On the gross section of compression members of truss and lattice girders with pin connections for axial stress,

$8 \left(1 - 0.005 \frac{l}{r}\right)$ tons per square inch

where l = the greatest length of the unbraced portion of the member and r = the least radius of gyration,

provided that, in truss and lattice girders, the working compressive stress on the gross section shall in no case exceed 6.8 tons (15,232 lb.) per square inch.

For parts in shear:—

On the gross section of web plates, 5 tons (11,200 lb.) per square inch.

On shop rivets, turned tight-fitting bolts and pins, 6 tons (13,440 lb.) per square inch.

For bearing areas:—

On shop rivets, turned tight-fitting bolts and pins, 12 tons (26,880 lb.) per square inch.

NOTE.—For connections which are to be made in the field or where black bolts are used instead of rivets an excess of 15 per cent. in the case of field rivets and 20 per cent. in the case of black bolts over the number required according to the above working stresses, both for shearing and bearing, shall be provided.

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For pins subjected to bending:—

On the extreme outer fibres,

12 tons (26,880 lbs.) per square inch.

Where structural steel of the milder quality specified in Clause I, Part I, of the B. S. Specification No. 153 of 1922 for material to be pressed cold is used, the foregoing working stresses where applicable shall be reduced by 10 per cent.

42. *Rollers.*—The pressure in tons per lineal inch on rollers of rolled steel shall not exceed $0.25d$, where “d” is the diameter of the roller in inches, but no roller shall be of less diameter than 4 inches.

43. *Working Stresses in Special Steels.*—Where special steels having an ultimate tensile strength higher than that of the structural steel of “A” quality specified in Clause I, Part I, of the B. S. Specification No. 153 of 1922 are used under a satisfactory guarantee, the working stresses for such special steels may be increased above those herein specified as shall be determined by the Engineer.

44. *Working Stresses in Cast Steel.*—The working stresses in cast steel in bearings, after allowing for the Impact effect of the live load, shall not exceed the working stresses herein specified for structural steel of “A” quality.

45. *Working Stresses in Wrought Iron.*—Where wrought iron is used, the working stresses in any member shall not exceed 75 per cent. of the corresponding tensile working stresses, or 85 per cent. of the corresponding compressive working stresses, herein specified for structural steel of “A” quality. For shear stresses in wrought iron take 80 per cent. of the corresponding stress for steel.

46. *Working Stresses in Cast Iron.*—Cast iron shall not be used in any portion of the structure of a bridge carrying a railway except only when subject to direct compression, but may be used in other bridges where subject to bending. The working stresses in the material shall not exceed 2.5 tons (5,600 lbs.) per square inch in tension and 10 tons (22,400 lbs.) per square inch in compression, after allowing for the Impact effect of the live load.

47. *Copper Alloy Bearings.*—Where hard copper alloys are used for sliding bearings, the pressure thereon shall not exceed 2 tons (4,480 lbs.) per square inch after allowing for the Impact effect of the live load.

48. *Area of Bearings or Bed plates.*—The area of bearings or bed plates shall be so proportioned that the pressure on the material forming the bed stones, due to the combined dead load, live load with Impact, and centrifugal effect, if any, shall not exceed the following limits:—

On granite per sq. ft. 25 tons (56,000 lbs.)

On sand stone or similar per sq. ft. 20 tons (44,800 lbs.)

On cement per sq. ft. 18* tons (40,320 lbs.)

Concrete (4 to 1) }

Concrete (6 to 1) } per sq. ft. 15* tons (33,600 lbs.)

These limits may, at the discretion of the Engineer, be exceeded by 20 per cent. when the maximum combination of loads specified in Rule 37 is taken into account.

49. *Frictional Coefficients for Expansion Bearings.*—Where the frictional resistance of the expansion bearings has to be taken into account, the following coefficients shall be assumed in calculating the amount of friction on the bearings:—

For roller bearings 3 per cent.

For sliding bearings of steel on hard copper alloy bearings . 15 „

For sliding bearings of steel on cast iron or steel „ 25 „

* These loads are specified on the assumption that the cement concrete forming the bed stones shall be made from Portland Cement complying with the current edition of the British Standard Specification for Portland Cement (No. 12).

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50. For expansion and contraction of the structure, due to variations of temperature, under dead load, the friction on one expansion bearing shall be considered as an additional stress throughout the chord to which the bearing plates are attached.

In those cases where the abutments are entirely rigid, the friction on the bearings produced by live load (without Impact) may be considered as a relief of stress, uniform throughout the chord to which the bearing plates are attached.

51. *B. S. Structural Steel Sections.*—The whole of the rolled steel sections used in the work shall be in accordance with the current edition of Report No. 6 Dimensions and Properties of British Standard Rolled Steel Sections for Structural Purposes.

52. *Effective Spans and Lengths.*—For the purpose of calculating the bending moments, stresses, shears and working strengths, the effective spans and lengths shall be taken as follows:—

For main girders the effective span shall be the distance between centres of bearing plates or rocker pins.

For cross girders the effective span shall be the distance between the centres of the main girders or trusses.

For rail or road bearers the effective span shall be the distance between the centres of the cross girders.

Where a cross girder or bearer terminates on an abutment or pier, the centre of the bearing thereon shall be taken as one end of the effective span.

For web compression members:—

(a) Where the web consists of a single system of triangulation, the effective length shall be the distance between the centres of gravity of the upper and lower booms measured along the axis of the compression member.

(b) Where the web consists of more than one system, the effective length shall be taken

- (i) between two points of intersection of the web members,
- (ii) between one point of intersection and the centre of gravity of the boom, or
- (iii) between the centres of gravity of the upper and lower booms,

whichever gives the greatest value of $\frac{l}{r}$ of the member,

where l = the effective length

and r = the least radius of gyration.

For compression booms and end posts of truss girders, the effective length shall be taken in the weakest plane of bending either between the points of intersection of the vertical or lateral bracing with the booms or end posts, or between the points at which rigidly connected cross girders rest on the booms. Where there is no lateral bracing between the booms, the effective length shall be taken as three-quarters of the length of the boom from centre to centre of the tops of the end posts, unless it is efficiently bracketted to the cross girders to give it adequate support, when the effective length shall be measured between the brackets.

For bending in pins the effective span shall be the centre of bearings unless the pins have sleeves designed to distribute the loads.

53. *Effective Depths.*—The effective depth of rivetted plate or truss girders shall be taken as the distance between centres of gravity of the upper and lower booms.

54. *Sectional Areas.*—The effective nett sectional area shall be taken for all tension members. This area shall be the least that can be determined

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from any plane or planes cutting each component plate or section either perpendicularly to its axis, diagonally, or following a zig-zag line through adjacent rivet holes. From the gross sectional area in each case the cross sectional area of the plate or section cut out by the intersecting holes shall be deducted. Where any portion of the sectional area is measured along a diagonal plane, four-fifths only of the nett area of such portion shall be taken in computing the effective area with a minimum equal to that obtained by assuming all the holes to be in one perpendicular plane.

55. The gross sectional area shall be taken for all compression members.

56. The shearing stress on the web plates of plate girders shall be calculated on the gross sectional area of the full depth of the plate.

57. For rolled beams and channels, the gross sectional area of the web resisting shearing stress shall be calculated on the full depth of the beam or channel.

58. As an alternative Waddell's method of determining the nett section of tension members may be used: *vide* Technical Paper No. 211, Third Report of I. R. B. C.

59. *Symmetry of Sections.*—All sections shall as far as possible be symmetrical about the line of resultant stress, and all rivets grouped symmetrically about the same line. The neutral axes of intersecting main members shall meet in a common point.

60. *Minimum Sections.*—No plate or bar less than five-sixteenths ($\frac{5}{16}$) of an inch in thickness shall be used in the main members of the bridge when both sides are accessible for painting, nor less than three-eighths ($\frac{3}{8}$) of an inch when only one side is accessible except where it is rivetted to another plate or bar. In floor plates and parapets a minimum thickness of one-quarter ($\frac{1}{4}$) of an inch may be used.

61. No angle less than three (3) inches by two (2) inches shall be used for the main members of girders or trusses.

62. No angle less than two and a half ($2\frac{1}{2}$) inches by two (2) inches nor flat bar less than two (2) inches shall be used in any part of a bridge structure, except for hand railing.

63. End angles connecting longitudinal bearers to cross girders or cross girders to main girders shall be not less in thickness than three-quarters of the thickness of the web plates of the longitudinal bearers and cross girders respectively.

64. *Rolled Sections.*—The transverse strength of rolled sections shall be based upon their section moduli.

65. *Clevises and Turnbuckles.*—Clevises and turnbuckles shall in all cases develop the full strength of the bars of which they form a part.

PLATE GIRDERS.

66. *Flanges and Webs.*—In plate girders the flange plates and flange angles shall be calculated as resisting the whole of the bending stresses, and web plates as resisting the whole of the shearing stresses, but one-eighth of the web plates may be included in the estimated sectional area of each of the flanges if the web plates are efficiently covered at joints to transmit the horizontal stresses.

67. *Flange Rivets.*—The flanges of plate girders shall be connected to the web plate by sufficient rivets through the flange angles to transmit the horizontal shearing force, combined with any vertical wheel loads, including Impact effect, that are directly applied to the flange. The computed stress on any rivet shall be the resultant stress due to the horizontal shear and the vertical shear from the wheel load.

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68. The horizontal shearing force on the rivets per unit of length at any section may be calculated by the formula :—

$$F = \frac{V}{d} \times \frac{a}{a - \frac{W}{8}}$$

where F = horizontal shearing force per unit of length,

V = total vertical shearing force at section being considered,

d = depth between centres of top and bottom rows of rivets,

a = gross sectional area of flange neglecting portion of web included in area of flange,

and W = gross sectional area of web plate.

69. Where railway sleepers rest directly upon the flanges, each wheel load shall be assumed to be uniformly distributed over a distance equal to twice the pitch of the sleepers with a maximum of four (4) feet.

70. *Flange Section.*—Flange angles shall form as large a part of the area of the flange as practicable and the number of flange plate shall be reduced to a minimum. To obtain an even distribution stress over the cross section of the flange plates, they shall not project beyond the outer line of rivets which pass through the flange angles more than sixteen times their thickness, or eight (8) inches, whichever dimension is the smaller. The unsupported length of the compression flange between effective stiffeners or side brackets rivetted to deep cross girders in the case of half through spans, or of the cross frames in the case of open deck spans, should preferably not exceed fifteen times its width, nor should the total length of the flange exceed forty times its width unless effective lateral stays are provided. At the end of each flange plate, rivets shall be provided at a pitch not exceeding four and a half diameters equal in strength to that of the flange plate, and the part of the flange plate extending beyond its theoretical end shall contain one-half of these rivets.

71. *Web Stiffeners.*—Web plates shall have stiffeners rivetted on both sides at the ends and inner edges of the bearing plates, and at all points of local and concentrated loads, also at points throughout the length of the girder, generally not farther apart than the depth of the girder, with a maximum spacing of six (6) feet, when the thickness of the web is less than one-sixtieth of the unsupported distance between the flange angles.

72. Stiffeners over the bearing plates shall have sufficient area to carry the entire shear without exceeding the specified intensity of working stress, while the intermediate stiffeners and the rivets connecting them to the web plates should be of sufficient area to take two-thirds of the vertical shear at the point of attachment. All stiffeners shall be proportioned as struts having a length equal to three-fourths of the depth of the girder.

73. *Lateral Bracing.*—In spans with open floors, rigid lateral diagonal bracing shall extend from end to end, of sufficient strength to transmit to the piers or abutments the lateral stresses due to wind pressure and centrifugal action.

74. The diagonal bracing shall, where possible, be rigidly secured to the rail or road bearers so as to transmit to the main girders the longitudinal thrust due to the tractive effort and braking effect, in order to relieve the cross girders of horizontal bending stresses.

75. In plated floors such bracing may be omitted where the plating is of sufficient strength to resist these forces.

RIVETTED TRUSS OR LATTICE GIRDERS.

76. *Booms and End Posts.*—The top booms and end posts shall preferably be of trough section suitably stiffened at the edges. Both top and bottom

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booms shall also be stiffened where necessary by diaphragms, tie plates, or latticing. The overall width of top booms should preferably be not less than one-fifteenth of the unsupported distance between points of intersection of the lateral bracing, or of substantial side brackets where the lateral bracing is omitted, nor should the total length of the boom exceed forty times its width unless effective lateral stays are provided.

77. *Web Tension Members.*—Web tension members shall preferably be of rigid construction, but they may be flat bars, except near the centres of girders, where they shall be of rigid construction. Counterbracing, where employed, shall be of similar construction to the centre tension members. In order to reduce vibration, distance pieces shall be used between the plates or flat bars forming long tension members.

78. Where angle bars connected by one leg are used as tension members, the effective sectional areas shall be taken as that of the rivetted leg added to one-half of the free leg.

79. *Compression Members.*—In no case shall a compression member have a greater length than one hundred and twenty times its least radius of gyration, or forty-five times its least width. The open sides of long compression members shall be stayed with intermediate tie plates or latticing.

80. Where the component parts of a member are laced together to form a unit, the ratio $\frac{l}{r}$ for any component part between the connections of the latticing shall be appreciably less than this ratio for the member as a whole.

81. To obtain an even distribution of stress over the cross section, the outstanding legs of compression members shall not project beyond the outer line of rivets which pass through the flange angles more than sixteen times their thickness or eight (8) inches, whichever dimension is the smaller, unless suitably stiffened at the edges. The unsupported width at right angles to the line of resultant stress should not exceed forty times its thickness, and any excess over this width shall not be included in the effective sectional area.

82. The tie plates shall have a thickness of not less than one-fiftieth of their unsupported width, except where they are stiffened with angle bars on their edges, when they may be five-sixteenths ($\frac{5}{16}$) of an inch in thickness. The length of the tie plates at the ends of laced struts shall be not less than the vertical side plates of the main booms, and shall be at least equal to the depth of the cross girders where these are directly attached to the struts.

83. At the ends of rivetted columns or struts for a length equal to at least one and a half times the width of the member, the pitch of rivets shall not exceed four and a half diameters.

84. *Latticing.*—The latticing of compression members shall be proportioned to resist a transverse shear at any point in the length of the member equal to at least $2\frac{1}{2}$ per cent. of the axial stress in the member, which shear shall be considered as divided equally among all transverse stiffening systems in parallel planes whether of continuous plates or of latticing.

The minimum width of lattice bars shall be:—

Two and a half ($2\frac{1}{2}$) ins. for seven-eighth ($\frac{7}{8}$) in. rivets.

Two and a quarter ($2\frac{1}{4}$) ins. for three-quarter ($\frac{3}{4}$) in. rivets.

Two (2) ins. for five-eighth ($\frac{5}{8}$) in. rivets.

85. The minimum thickness of the lattice bars shall be not less than one-fortieth of the shortest distance between the centres of rivets in the case of single latticing, and one-sixtieth of this distance for double latticing, rivetted at the intersections. Rolled sections of equivalent strength may be used instead of flats.

86. Lattice bars should generally be inclined at an angle of about 60 degrees to the axis of the member when single latticing is used, and at an

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angle of about 45 degrees with double latticing, furthermore, the maximum spacing of lattice bars shall be such that the ratio $\frac{l}{r}$ for the portions between consecutive connections of the latticing shall be appreciably less than this ratio for the member as a whole.

87. *Lateral Bracing*.—In spans with open floors, rigid lateral diagonal bracing shall extend from end to end, of sufficient strength to transmit to the piers or abutments the lateral stresses due to wind pressure and centrifugal action.

88. The diagonal bracing shall, where possible, be rigidly secured to the rail or road bearers so as to transmit to the main girders the longitudinal thrust due to the tractive effort and braking effect, in order to relieve the cross girders of horizontal bending stresses.

89. In plated floors such bracing may be omitted where the plating is of sufficient strength to resist these forces.

90. Where the depth permits, lateral diagonal bracing shall be fixed between the top booms of main girders of sufficient rigidity to maintain the booms in line and of sufficient strength to transmit the wind pressure to the portal bracing between the end posts.

91. *Overhead Cross Bracing between Struts*.—The overhead cross bracing between struts shall be proportioned to carry at least 50 per cent. of the panel load due to wind, and the struts shall be calculated to resist the bending stresses from the wind loads.

92. *Portal Bracing*.—Portal bracing of the maximum depth permissible with the required head-room shall be rivetted to the end posts. Rigid knee brackets shall be rivetted to the portal bracing and end posts. In determining the sectional area of the end posts, provision must be made for the bending stresses due to the wind pressure. The end posts may be considered as fixed at the ends.

93. *Spacing and Depth of Trusses*.—The width between centres of main trusses should be sufficient to resist overturning with the specified wind pressures and loading conditions, otherwise provision must be specially made to prevent this.

94. In no case shall this width be less than one-twentieth of the effective span, nor shall the depth of the trusses be greater than three times the width between centres of trusses.

95. In special cases it may be necessary to depart from the limits laid down in this clause, but each such case requires special sanction.

96. *Joints*.—The butting ends of all spliced members, whether in tension or compression, shall be fully covered and rivetted to develop the effective strength of the member.

97. Web joints shall have double covers of adequate width to admit of sufficient rivets to transmit the whole of the shearing stress at the joint. Where a portion of the web is included as flange section, the web covers and their connecting rivets shall be proportioned to transmit bending as well as shearing stresses.

98. The sectional area shall be not less than 5 per cent. in excess of the section covered in the case of symmetrical covers and 10 per cent. in the case of unsymmetrical covers.

99. The centre of gravity of the covers shall coincide as nearly as possible with the centre of gravity of the section covered.

RIVETTING.

100. *Effective Diameter of Rivets*.—In calculating the number of rivets required the rivet hole shall be adopted as the standard of dimension of the

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finished rivet,* the diameter of the finished rivets, as marked on the drawings, being taken as the effective diameter for calculating area.

101. The shearing area of rivets, bolts or pins in double shear shall be calculated as having twice the shearing strength of those in single shear.

102. The effective bearing area of a pin, rivet or fitted bolt shall be the diameter multiplied by the thickness of the member or unit transmitting or receiving the stress, except that for rivets or bolts with countersunk heads one-half of the depth of the countersink shall be omitted. In the case of black bolts three-quarters of the diameter shall be taken.

103. *Nett Section at Rivet Holes.*—In deducting for snapheaded rivets or for bolts, the diameter of the hole shall be taken as the diameter of the finished rivet or bolt as marked on the drawings, and as one-eighth ($1/8$) of an inch larger for holes countersunk in the outer plate or section.

104. *Minimum Pitch of Rivets.*—The distance between centres of rivets shall be not less than three times the diameter of the rivet.

105. *Maximum Pitch of Rivets.*—In built tension members the rivet pitch shall not exceed sixteen times the thickness of the thinnest outside plate or angle, except in angles having two lines of staggered rivets, where the pitch on each line may be twice this limit, but it shall not exceed twelve (12) inches in any case. In built compression members, the rivet pitch shall not exceed twelve times the thickness of the thinnest outside plate or angle, except in angles having two lines of staggered rivets where the pitch be one and a half times this distance.

106. *Edge Distance of Rivets.*—The minimum distance from the centre of any rivet to a sheared edge shall be:—

One and three-quarters ($1\frac{3}{4}$) ins. for one (1) in. rivets.

One and a half ($1\frac{1}{2}$) ins. for seven-eighth ($7/8$) in. rivets.

One and a quarter ($1\frac{1}{4}$) ins. for three-quarter ($3/4$) in. rivets.

One and an eighth ($1\frac{1}{8}$) ins. for five-eighth ($5/8$) in. rivets.

and, to a rolled or planed edge:

One and a half ($1\frac{1}{2}$) ins. for one (1) in. rivets.

One and a quarter ($1\frac{1}{4}$) ins. for seven-eighth ($7/8$) in. rivets.

One and an eighth ($1\frac{1}{8}$) ins. for three-quarter ($3/4$) in. rivets.

One (1) in. for five-eighth ($5/8$) in. rivets.

107. Where two or more flange plates are employed, the edge distance from the centre line of the nearest rivet shall be not greater than eight times the thickness of the thinnest outside plate.

108. *Rivets through Packings.*—Rivets carrying calculated stress and passing through packings over three-eighths ($3/8$) of an inch in thickness shall be increased by at least 20 per cent. over the nett number required. The additional rivets shall preferably be placed outside one of the connected members.

GENERAL.

109. *Camber.*—The camber shown on the drawings indicates the camber which the main girders are to have when supported at end bearings only with full dead load on. The camber of trusses may be obtained by making the horizontal component of the panel length of the top boom longer than the horizontal component of the corresponding panel length of the bottom boom. A camber diagram shall be shown on the erection drawings indicating the relative height of each panel point when erected on blocks at the Contractor's works.

* It is recommended that wherever possible the diameters of the rivets and bolt holes should be stated in odd sixteenths of an inch so that the diameters of rivets and bolts, when ordered to the permissible clearances, may be in eighths of an inch to suit present ordinary manufacture.

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(c)—SPECIAL RULES FOR TIMBER GIRDERS AND TRESTLES.

30. *Impact*.—The increment for Impact for timber girders and trestles shall be the same as that allowed for steel and iron girders and trestles: see rules 26, 27 and 28 and the Notes (a) and (b) below them.

31. *Maximum permissible stresses*.—Must be not greater than $1/5$ th of the ultimate strength of the material used.

It must be remembered that timber in railway bridges may be subjected to stresses of the following kinds, (1) tensile, (2) compressive, (4) shearing, and (4) bearing; also that the ultimate strength of timber “across the grain” is usually different from that of the same timber “with the grain”.

Unit compressive stresses must be reduced in accordance with a suitable “column formula”.

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110. *Open Sections and Drain holes.*—All structures shall be designed so that all parts will be accessible for inspection, cleaning and painting. Drain holes shall be provided at all places where pockets or depressions are likely to hold water.

111. *Provision for Temperature and Deflection.*—Where provision for expansion and contraction due to changes of temperature is necessary, it shall be provided to the extent of not less than one (1) inch for every hundred (100) feet of length.

112. The expansion bearings shall be so designed as to permit of inspection and lubrication. They shall allow free movement in a longitudinal direction and at the same time prevent any transverse motion.

113. Where the span exceeds one hundred (100) feet in length, bearings shall be provided at both ends of the main girders to permit deflection of the girders without unduly loading the face of the abutment or pier.

114. *Provision for Corrosion.*—Where specified by the Engineer (or the Purchaser), an addition shall be made to the sectional areas required to resist the computed stress so as to allow for corrosion when the bridge is situated where the climatic influences or local conditions are such that corrosion is set up in the steel work.

115. *Loads.*—All dead loads shall be assumed to be evenly distributed in the case of plate girders, except the load brought upon the main girders from the cross girders when they exceed ten (10) feet centres. In the case of lattice or truss girders, the dead load shall be assumed to be collected at the panel points.

116. *Drawings.*—Three sets of prints on linen of the drawings of the steelwork as actually manufactured and of the erection and marking drawings shall be supplied by the Contractor, together with one set of unmounted photographs of such parts of the work as are temporarily erected in the Contractor's yard, and of any special details of the work.

(c) SPECIAL RULES FOR TIMBER GIRDERS AND TRESTLES.

117. *Impact.*—The Impact Factor $\frac{120}{90 + \frac{n+1}{2}L}$ which is described in sub-clause 5(a) of Part 3 of the British Standard Specification shall apply to train-loads on spans up to 50 feet only.

118. For spans over 50 feet, the increment for Impact is to be calculated, for Timber girders and Trestles under railways of all gauges, by multiplying the train-load by the factor $\frac{300}{300 + L}$.

119. For moving loads on a road or foot bridge over a railway, or on the road-way or footways of a combined road and railway bridge, the Impact Factors for spans up to, and over, 50 feet shall be respectively $\frac{1}{2}$ of

$$\frac{120}{90 + \frac{n+1}{2}L} \text{ and } \frac{1}{2} \text{ of } \frac{300}{300 + L}.$$

NOTE.—The Railway Board are willing to consider recommendations for smaller Impact Factors than those prescribed in rules 117–119 in any particular case, such as that of a well balanced electric or steam locomotive, provided that proof is produced that the factors prescribed in rules 117–119 are excessive for the particular case in question.

120. *Maximum permissible stresses.*—Must be not greater than 1/5th of the ultimate strength of the material used.

It must be remembered that timber in railway bridges may be subjected to stresses of the following kinds, (1) tensile, (2) compressive, (3) shearing, and (4) bearing; also that the ultimate strength of timber “across the grain” is usually different from that of the same timber “with the grain.”

Unit compressive stresses must be reduced in accordance with a suitable “column formula”.

EXISTING BRIDGE RULES, 1923.

(d)—SPECIAL RULES FOR ARCHED MASONRY BRIDGES.

(i) *Methods of Calculating Stresses.*

32. The elastic theory, usually adopted in the case of steel and ferro-concrete arches, is not strictly applicable in the case of the ordinary masonry arch built under a railway. For calculating stresses in arched masonry bridges, the ordinary methods described in standard text-books may be followed. The ordinary method of locating the lines of pressure is based upon assumptions which cannot be verified, but which have been proved by long experience to be sufficiently accurate for all practical purposes.

33. *Impact.*—Pending the evolution of more exact formulæ [see Note (a) under rule 28, above] the increment for Impact to be allowed on the whole of the moving load (m) may be calculated, for spans up to and above 50 feet respectively, from the formulæ

$$\frac{1}{2} \text{ of } \frac{120}{90 + \frac{n+1}{2} L} M \text{ and } \frac{1}{2} \text{ of } \frac{300}{300 + L} M$$

if there is a cushion of earth, ballast, etc., more than two feet thick below the bottom of the sleepers.

34. If, however, the thickness of the cushion is not more than two feet, the increments for Impact shall [subject to the relaxation of this rule in circumstances described in Note (b) under rule 28] be calculated from the formulæ

$$\frac{120}{90 + \frac{n+1}{2} L} M_T \text{ and } \frac{300}{L + 300} M_T$$

in the case of the train load (M_T) and from the formulæ

$$\frac{1}{2} \text{ of } \frac{120}{90 + \frac{n+1}{2} L} M_R \text{ and } \frac{1}{2} \text{ of } \frac{300}{300 + L} M_R$$

in the case of load (M_R) moving on a road or foot bridge over a railway or on the roadway or footways of a combined road and railway bridge.

35. *Transmission of Train Load to Arch.*—The area of the arch over which the load is distributed may be taken as bounded by lines drawn at a slope of $\frac{1}{2}$ horizontal to 1 vertical from the bottom edges of the sleepers to the top of the arch ring.

(ii) *Maximum Permissible Stresses.*

36. Maximum permissible stresses must be not greater than $1/5$ th of the ultimate strength of the material used.

The co-efficient of friction of a mortar joint may be taken as .7. The resistance in shear of a lime-mortar joint may be taken at not more than 300 lbs. per square foot.

(e) SPECIAL RULES FOR FERRO-CONCRETE BRIDGES.

(i) *Methods of Calculating Stresses.*

37. Following the procedure adopted by the Public Works Department in most of the provinces of India (*see* Technical Paper No. 191) the Railway Board do not desire to bind designers to follow the methods of calculating stresses described in any particular text-book.

38. In calculations submitted to Government Inspectors of Railways the notation adopted should be that published by the Joint Committee (on reinforced concrete) of the Royal Institute of British Architects.

PROPOSED BRIDGE RULES.

(d)—SPECIAL RULES FOR ARCHED MASONRY BRIDGES.

(i) *Methods of Calculating Stresses.*

121. The elastic theory, usually adopted in the case of steel and ferro-concrete arches, is not strictly applicable in the case of the ordinary masonry arch built under a railway. For calculating stresses in arched masonry bridges, the ordinary methods described in standard text-books may be followed. The ordinary method of locating the lines of pressure is based upon assumptions which cannot be verified, but which have been proved by long experience to be sufficiently accurate for all practical purposes.

122. *Impact.*—Pending the evolution of more exact formulæ [see Note under rule 119 above] the increment for Impact to be allowed on the *whole* of the moving load (*m*) may be calculated, for spans up to and above 50 feet respectively, from the formulæ

$$\frac{1}{2} \text{ of } \frac{120}{90 + \frac{n+1}{2} L} M \text{ and } \frac{1}{2} \text{ of } \frac{300}{300 + L} M$$

if there is a cushion of earth, ballast, etc., more than two feet thick below the bottom of the sleepers.

123. If, however, the thickness of the cushion is not more than two feet, the increments for Impact shall [subject to the relaxation of this rule in circumstances described in Note under rule 119] be calculated from the formulæ

$$\frac{120}{90 + \frac{n+1}{2} L} M_T \text{ and } \frac{300}{L + 300} M$$

in the case of the train load (M_T) and from the formulæ

$$\frac{1}{2} \text{ of } \frac{120}{90 + \frac{n+1}{2} L} M_R \text{ and } \frac{1}{2} \text{ of } \frac{300}{300 + L} M_R$$

in the case of load M_R moving on a road or foot bridge over a railway or on the roadway or footways of a combined road and railway bridge.

124. *Transmission of Train Load to Arch.*—The area of the arch over which the load is distributed may be taken as bounded by lines drawn at a slope of $\frac{1}{2}$ horizontal to 1 vertical from the bottom edges of the sleepers to the top of the arch ring.

(ii) *Maximum Permissible Stresses.*

125. Maximum permissible stresses must be not greater than $\frac{1}{5}$ th of the ultimate strength of the material used.

The co-efficient of friction of a mortar joint may be taken as 7. The resistance in shear of a lime-mortar joint may be taken at not more than 300 lbs. per square foot.

(e) SPECIAL RULES FOR FERRO-CONCRETE BRIDGES.

(i) *Methods of Calculating Stresses.*

126. Following the procedure adopted by the Public Works Department in most of the provinces of India (see Technical Paper No. 191) the Railway Board do not desire to bind designers to follow the methods of calculating stresses described in any particular text-book.

127. In calculations submitted to Government Inspectors of Railways the notation adopted should be that published by the Joint Committee (on reinforced concrete) of the Royal Institute of British Architects.

EXISTING BRIDGE RULES, 1923.

39. For guidance in the duties imposed on him by sections 19 and 20 of the Indian Railways Act of 1890, a Government Inspector should consult the Second Report of the Joint Committee mentioned in rule 38.

40. The essential conditions in ferro-concrete designs are (1) that the modulus of elasticity of concrete is to be assumed to remain constant within the limits of the working stress, and (2) that all tensile stresses shall be taken by the steel reinforcement.

41. *Impact*.—The increment for impact for ferro-concrete bridges shall be the same as that allowed for steel and iron bridges (see rules 26—28 and the notes below them) except in the cases of arches and box culverts where the moving load is applied to the concrete through a substantial earth cushion, when the increment for Impact may be the same as that allowed for arched masonry bridges (in rule 33).

(ii) *Maximum permissible stresses.*

42. Concrete in compression must not be subjected to a greater total working load (see rules 6 and 13) than one-quarter of its ultimate crushing load, which must be ascertained by tests of the actual concrete used for each bridge: subject to this condition, the following stresses are permissible:—

Concrete in compression in beams	600 lbs. per square inch.
Concrete in columns	450 ditto.
Concrete in beams in pure shear uncombined with diagonal tension.	120 ditto.
Adhesion of concrete to metal	100 ditto.
Steel in tension	16,000 ditto.
Iron in tension	12,000 ditto.
Steel in compression	15 times the intensity of the stress in the surrounding concrete.
Iron in compression	13 times the intensity of stress in the surrounding concrete.
Steel in shear	12,000 lbs. per square inch.
Iron in shear	9,000 ditto.
For concrete not weaker than 1 : 2 : 4 and steel reinforcement.	$n = 15$
For concrete not weaker than 1 : 2 : 4 and wrought iron reinforcement.	$n = 13$

(n =ratio of modulus of elasticity of steel or iron in tension to modulus of elasticity of concrete in compression).

43. The figures relating to concrete are based on an ultimate crushing strength of 2,400 lbs. per square inch in six inch cubes at 28 days for a mixture corresponding approximately to a 1 : 2 : 4 concrete: for other ultimate crushing strengths the stresses are to be correspondingly modified.

44. Cement must be not inferior to that defined in the British Standard Specification for Portland Cement. Cement must be measured by weight; for gauging purposes a cubic foot of cement must not weigh less than 95 lbs.; for instance, a 1 : 2 : 4 concrete means 95 lbs. of cement to 2 cubic feet of sand and 4 cubic feet of aggregate.

(f).—APPLICABILITY OF THE RULES TO EXISTING BRIDGES.

45. The rules given above are intended primarily for the guidance of Inspectors of Railways in their inspections of new railways, or of new works on existing railways, prior to opening; and no bridges which do not come up to the standard thus prescribed may be passed without restriction, except with the sanction of the railway Board, though it is open to the Inspector to

PROPOSED BRIDGE RULES.

128. For guidance in the duties imposed on him by sections 19 and 20 of the Indian Railways Act of 1890, a Government Inspector should consult the Second Report of the Joint Committee mentioned in rule 127.

129. The essential conditions in ferro-concrete designs are (1) that the modulus of elasticity of concrete is to be assumed to remain constant within the limits of the working stress, and (2) that all tensile stresses shall be taken by the steel reinforcement.

130. *Impact.*—The increment for Impact for ferro-concrete bridges shall be the same as that allowed for Timber girders and Trestles (see rules 117—119 and the note below them) except in the cases of arches and box culverts where the moving load is applied to the concrete through a substantial earth cushion, when the increment for Impact may be the same as that allowed for arched masonry bridges (in rule 122).

(ii) *Maximum permissible stresses.*

131. Concrete in compression must not be subjected to a greater total working load (see rules 6 and 13) than one-quarter of its ultimate crushing load, which must be ascertained by tests of the actual concrete used for each bridge: subject to this condition, the following stresses are permissible:—

Concrete in compression in beams	600 lbs. per square inch.
Concrete in columns	450 ditto.
Concrete in beams in pure shear uncombined with diagonal tension.	120 ditto.
Adhesion of concrete to metal	100 ditto.
Steel in tension	16,000 ditto.
Iron in tension	12,000 ditto.
Steel in compression	15 times the intensity of the stress in the surrounding concrete.
Iron in compression	13 times the intensity of stress in the surrounding concrete.
Steel in shear	12,000 lbs. per square inch.
Iron in shear	9,000 ditto.
For concrete not weaker than 1 : 2 : 4 and steel reinforcement.	$n=15$
For concrete not weaker than 1 : 2 : 4 and wrought iron reinforcement.	$n=13$

(n =ratio of modulus of elasticity of steel or iron in tension to modulus of elasticity of concrete in compression).

132. The figures relating to concrete are based on an ultimate crushing strength of 2,400 lbs. per square inch in six inch cubes at 28 days for a mixture corresponding approximately to a 1 : 2 : 4 concrete: for other ultimate crushing strengths the stresses are to be correspondingly modified.

133. Cement must be not inferior to that defined in the British Standard Specification for Portland Cement. Cement must be measured by weight; for gauging purposes a cubic foot of cement must not weigh less than 95 lbs.; for instance, a 1 : 2 : 4 concrete means 95 lbs. of cement to 2 cubic feet of sand and 4 cubic feet of aggregate.

(f).—APPLICABILITY OF THE RULES TO EXISTING BRIDGES.

134. The rules given above are intended primarily for the guidance of Inspectors of Railways in their inspections of new railways, or of new works on existing railways, prior to opening; and no bridges which do not come up to the standard thus prescribed may be passed without restriction, except with the sanction of the Railway Board, though it is open to the Inspector to

EXISTING BRIDGE RULES, 1923.

make his recommendations as to whether such bridges may be used and, if so, under what conditions. (See Rules 1 to 5.)

46. The rules for calculating stresses apply also to the investigations of the strength of existing bridges; it is desired that when such investigations are submitted to the Railway Board, whether for new or old bridges, the methods prescribed above shall invariably be adopted, whether any other alternative method of calculation is also employed or not. (See Rule 5.)

47. In applying these rules for working stress to existing bridges under the actual loads of existing engines, the Inspector may, at his discretion, permit those bridges to be used for public traffic with stresses exceeding the limits laid down herein subject to such restrictions of speed, marshalling, periodical inspection, or other conditions, as may in his judgment be necessary in each case. He should report his action for the information of the Railway Board. *No such permission should be given unless the responsible officer of the railway concerned shall have certified that such usage will not involve danger to the traffic nor abnormal injury to the existing structures.*

48. If it is proposed to introduce a new type of engine or other rolling-stock, and it is found that the proposed rolling-stock will cause in existing bridges stresses exceeding the limits laid down in these rules, the Inspector should refer the matter to the Railway Board, with his recommendations on the subject, before the engines or other rolling-stock are ordered (See also Rule 5.)

NOTE.—When an Inspector has to exercise the powers vested in him by rule 47 or to submit recommendations to the Railway Board under rules 1—5, 45 and 48, he may with advantage refer to the reports of the Indian Railway Committee for guidance.



PROPOSED BRIDGE RULES.

make his recommendations as to whether such bridges may be used and, if so, under what conditions. (See Rules 1 to 5.)

135. The rules for calculating stresses apply also to the investigations of the strength of existing bridges; it is desired that when such investigations are submitted to the Railway Board, whether for new or old bridges, the methods prescribed above shall invariably be adopted, whether any other alternative method of calculation is also employed or not. (See Rule 5.)

136. In applying these rules for working stress to existing bridges under the actual loads of existing engines, the Inspector may, at his discretion, permit those bridges to be used for public traffic with stresses exceeding the limits laid down herein subject to such restrictions of speed, marshalling, periodical inspection, or other conditions, as may in his judgment be necessary in each case. He should report his action for the information of the Railway Board. *No such permission should be given unless the responsible officer of the railway concerned shall have certified that such usage will not involve danger to the traffic nor abnormal injury to the existing structures.*

137. If it is proposed to introduce a new type of engine or other rolling-stock, and it is found that the proposed rolling-stock will cause in existing bridges stresses exceeding the limits laid down in these rules, the Inspector should refer the matter to the Railway Board, with his recommendations on the subject, before the engines or other rolling-stock are ordered.

138. On old girders where a restriction of 10 miles per hour or less is rigidly enforced, no allowance for impact need be made, subject to the responsible authority certifying that the condition of the bridge and of the permanent way warrant this being done. The Transportation Department are to be held responsible that the restriction is rigidly observed. If the restriction is not observed the relaxation of the impact factor cannot be allowed and any permission given for increased loads on the strength of this relaxation must be at once cancelled.

139. Plate girder and rolled joist spans up to and including 40 feet clear span may be retained in use without speed restriction provided the calculated flange and web stresses with full impact allowance do not exceed the specified working stresses by more than 25 per cent. The bearing stresses on the rivets connecting web and flange if calculated by the B. E. S. A formula with full impact may be taken as 18 tons per square inch for steel and 15 tons per square inch for iron.

NOTE.—It is recommended that the Restriction Notice Board should shew the minimum time that must be taken between the restriction limits, and further that recording instruments should be used to detect infringements.

PROPOSED BRIDGE RULES.

APPENDIX A.

Method of determining the resultant out-of-balance "hammer blow" P_1 of any locomotive.

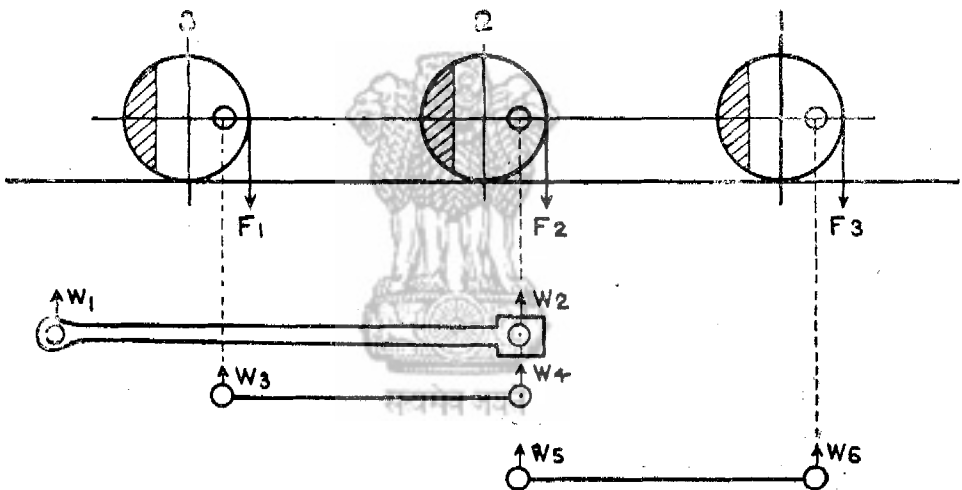
Formula.

$$P_1 = : 0.00065 M r.$$

Where $M r$ is the sum of the nett out-of-balance of all the wheels on one side of an engine with all the motion attached, expressed as a moment in inch lbs.

Shop method of determining nett out-of-balance $M r$.

A simple means of arriving at this figure is given in the diagram below, the forces F_1, F_2, F_3 , etc., being measured with a spring balance or otherwise. The axles should be mounted on knife edges for making the tests.



Example outside cylinder engine, coupling rods on same crank pin as connecting rod.

F_1, F_2, F_3 , = forces at tyre required to balance counterweight, crank pins, etc.

d = diameter of wheel over tyres in inches.

$\left. \begin{matrix} W_1 & W_2 \\ W_3 & W_4 \\ W_5 & W_6 \end{matrix} \right\}$ = weights of each end of connecting and coupling rods.

r = crank pin radius.

$$\text{Out-of-balance in 3} = F_1 \frac{d}{2} - W_3 r$$

$$\text{,, ,, ,, 2} = F_2 \frac{d}{2} - (W_2 + W_4 + W_5) r$$

$$\text{,, ,, ,, 1} = F_3 \frac{d}{2} - W_6 r.$$

$$\text{Total out-of-balance } M r = (F_1 + F_2 + F_3) \frac{d}{2} - (W_2 + W_3 + W_4 + W_5 + W_6) r.$$

PROPOSED BRIDGE RULES.

APPENDIX B (1).

Weights and Dimensions of various kinds of Roadway Loads.

Authority.		DIMENSIONS OF ONE UNIT.				MAXIMUM WEIGHT OF ONE UNIT.		AVERAGE WEIGHT.			REMARKS.
		OVER ALL.		BASE.				Per foot run of track.	PER SQUARE FOOT.		
		Length.	Width.	Wheel base.	Width of track.	Total.	Maximum on one wheel or foot.		Of base.	Of deck occu- pied.	
		Feet.	Feet.					lbs.	lbs.	lbs.	
PART A, FROM ROORKEE TREATISE, 1877, VOLUME III, PAGE 290, AND MILITARY ENGINEERING, 1902, PART III, PAGES 7 TO 10.	Unarmed men walk- ing freely.	3	1½	160 lbs.	160 lbs.	53	...	36	
	Unarmed men walk- ing freely crowded.	1½	1	160 "	160 "	133	...	133	
	Infantry in marching order in single file.	3	3	140 "	140 "	67	...	22	
	Infantry in marching order in file crowded.	...	6	280 "	280 "	50	...	133	
	Infantry in march- ing order in fours crowded.	...	8	560 "	560 "	70	...	133	
	Cavalry in single file marching.	12	6	1,400 "	...	116	...	20	
	Cavalry in half-sec- tions crowded.	12	8	2,800 "	...	392	...	50	
	Elephants, unloaded .	11	5	50 cwt.	30 cwt.	102	
	" loaded .	11	9	72 "	44 "	82	
	Camels, loaded .	10	7	15 "	10 "	112	...	24	
	Pack bullocks loaded .	5	2¾	5½ "	3½ "	79	...	45	
	Commissariat cattle crowded.	4 "	50	
	Pack Mules .	6	2½	10 "	6 cwt.	70	
	Mountain Battery Mules.	12	6	8 "	...	75	...	13	

Note.—Loaded camels require 10 feet clear width and 11 feet clear headway.
 Elephants " 12 " " " " 15 " " "

	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Tons.	Tons.	Cwt.	Lbs.	Lbs.
Steam rollers, 15 tons.	19 7	7 3	11 6	7 3	15	5	15	403	235
" " 20 "	20 3	8 3	11 8	8 0	20	6½	20	482	268
Traction engine, 11 tons.	18 4	7 10	10 7	...	11	...	12	305	176
Traction engine, 22 tons.	...	9 7	15 0	...	22	8	...	342	...
Tramcars, double, deck, four-wheeler.	27 5	7 0	6 0	5 0	11	3	9	893	137
Tramcars, double, deck, bogie.	33 6	7 1	13 6	5 0	14½	2¾	8¾	497	139
Tramcars, double, deck, bogie.	35 0	7 0	16 0	5 0	17¾	3¾	10	496	159
Tramcars, single deck, bogie.	35 0	7 0	17 0	5 0	15	3	8½	380	138
Boiler wagon	13 0	...	48	12	...	1,170	...
" "	22 0	...	38	9½
Stone "	...	6 8	9 0	...	16	4	...	600	...

Nature of Equipment.	DIMENSIONS AND WEIGHT OF ONE UNIT.					
	Overall width.	Wheel base.	Width of track.	Total weight Maximum of one unit.		
<i>Medium Artillery.</i>	Ft. In.	Ft. In.	Ft. In.	Cwt.	qr.	lb.
B. L. 6" 26 cwt. howitzer with carriage and limber.	7 10	...	6 7	82	2	22
B. L. 60-pr. gun with carriage and limber	8 0	15 9	6 7	127	...	14
B. L. 60-pr. ammunition wagon and limber	6 3½	8 3	5 3	48	...	12
<i>Field Artillery.</i>						
Q. F. 13-pr. gun with carriage and limber	6 3	9 11½	5 3	32	3	24
Q. F. 13-pr. ammunition wagon and limber	6 3	7 3½	5 3	32	3	23
Q. F. 18-pr. gun with carriage and limber	6 3	9 11	5 3	42	3	3
Q. F. 18-pr. ammunition wagon and limber	6 3	7 4¾	5 3	35	3	17
Q. F. 4½" howitzer with carriage and limber	6 3½	10 1½	5 3	41	3	...
Q. F. 4½" howitzer ammunition wagon and limber.	6 3½	8 7½	5 3	40	2	7
G. S. Wagon	...	7 0	5 3	47

PROPOSED BRIDGE RULES.

APPENDIX B (1)—*contd.*

MECHANICAL TRANSPORT.

Weights and dimensions of various types.

Autho- rity.	—	OVERALL.			BASE.		WEIGHT.		
		Length.	Width.	Height.	Wheel base.	Width of track.	F. A. W. unloaded.	R. A. W. unloaded.	Total unloaded.
		Ft. in.	Ft. in.	Ft. in.	Ft. in.	Ft. in.	Ton. Cwt.	Ton. Cwt.	Ton. Cwt.
PART D, FROM MILITARY ENGINEERING, VOLUME III, (1921).	3-ton lorry. . .	23 $\frac{1}{4}$	7 2	$\left\{ \begin{array}{l} 10 \ 2 \text{ Highest} \\ \ 0 \text{ point.} \\ \ 0 \text{ Top of} \\ \text{ Cab.} \end{array} \right\}$	14 3	6 10 $\frac{1}{2}$	1 17	2 9	4 6
	30-cwt. lorry . .	18 2	7 2	$\left\{ \begin{array}{l} 9 \ 10 \text{ Highest} \\ \text{ Point.} \\ \ 8 \ 2 \text{ Top of} \\ \text{ Cab.} \end{array} \right\}$	13 0	6 4 $\frac{1}{2}$	1 0	1 8	2 8 .
	15-cwt. van . .	15 6	6 1	6 10 Wind Screen.	11 6	6 1	0 15	1 0	1 15
	Heavy Ambulance .	16 6 $\frac{1}{2}$	6 6	8 6 Canopy.	11 6	6 1	0 19	1 1	2 0
	Workshop Lorry .	21 6	7 2	$\left\{ \begin{array}{l} 11 \ 3 \text{ Highest} \\ \text{ Point.} \\ \ 6 \ 10 \text{ Steering} \\ \text{ Wheel.} \end{array} \right\}$	14 3	7 2	2 9*	4 15*	7 4* *Loaded.
	Armoured Car (Lorry)	18 0	6 1	9 1	12 0	6 11	2 8	2 0	4 8

PART E, CALCULATED IN
TECHNICAL SECTION.

EQUIVALENT UNIFORM LOAD ON VARIOUS SPANS.

TOTAL LOADS ON SPAN IN TONS.

Span .	5	10	15	20	30	40	60	100	150 feet.
Line of Q. F. 4.7 inch guns drawn by 16 bullocks each.	8.6	8.6	8.8	9.0	9.6	10.4	11.4	16.0	22.5 tons.
Traction engine (20 tons) draw- ing train 5 cwt. per L. ft.	32.0	32.2	32.5	32.8	33.3	33.2	33.6	35.0	37.5 „

Note.—These equivalent loads apply directly to loads on longitudinal girders. For loads on cross girders, take half the load shown in the table for a span equal to double the distance between the cross girders.

XII

DIAGRAMS ACCOMPANYING THE FIRST INTERIM REPORT OF THE BRIDGE SUB-COMMITTEE 1925.

- Sheet No. 1. I. R. B. C. Tests. 1921.
Maximum Impact effects obtained by deflectometer.
- „ 2. I. R. B. C. Tests. 1921.
Critical frequencies—Calculated and observed.
- „ 3. I. R. B. C. Tests. 1921.
Comparison of Recorded and Calculated Impact.
- „ 4. A. R. E. A. Tests. 1908.
Comparison of Recorded and Calculated Impact.
- „ 5. G. I. P. Railway Tests. 1922.
Comparison of Recorded and Calculated Impact.
- „ 6. I. R. B. C. Tests. 1920 and 1921.
Comparison of Recorded and Calculated Impact for the lighter Indian
Engines.
- „ 7. Typical Engines on Indian Railways.
Calculated “ Hammer Blows.”
- „ 8. I. R. B. C. Tests. 1921.
Typical Deflectometer Diagrams.
- „ 9. Calculated Impact Percentage on Full Capacity Loads. Summary
covering I. R. B. C., G. I. P. & A. R. E. A. Tests. Modified for
Indian Standard Engine.
- „ 10. Impact on capacity loads and Covering Curve for Design Purposes.
- „ 11. Comparison of Impact Curves.
- „ 12. Typical Deflection Diagrams at 2nd critical speed.
- „ 13. A. R. E. A. Tests. Typical Deflection Diagrams shewing Engine
Effect and Effect of Wagons.
- „ 14. Example shewing a case where the Wagon Effect is greater than the
Engine Effect at the actual test speed, but less than the Engine Effect
at critical speed.
- „ 15. A. R. E. A. Typical Deflection Diagrams at critical speed.

Maximum Impact Effects obtained by Deflectometer.

Effective span in feet.	Name of Bridge and Type of Engine.	TEST SPEED CORRESPONDING TO COLUMN (6).		DEFLECTION AT CENTRE OF SPAN IN INCHES.			Impact per cent. Col. (7) \times 100. Col. (5)	REMARKS.
		M. P. HR.	Revs. per Sec. of Drivers.	At Crawl. Speed (mean value).	Maximum deflection.	Added deflection. Col. (6) — Col. (5).		
1	2	3	4	5	6	7	8	9
16-16	Jandiala (H. G. Engine)	19-8	1-96	.095	.105	.01	10-5	The actual tests on this span included speeds up to 53-4 M. P. HR. at which speed the impact was 8-4 per cent. See Record Sheets Nos. 55, 56.
33-33	Jandiala (H. G. Engine)	53-4 49-6	5-29 4-91	.210 .222	.275 .290	.065 .068	31-0 30-6	1st Series. See Record Sheets Nos. 49, 50. 2nd Series. See Record Sheets Nos. 52, 53.
41-33	Jandiala (H. G. Engine)	26-2 49-6	2-59 4-91	.246	.270	.024	9-8	Same impact registered at three different speeds. See Record Sheets Nos. 46, 47.
56-66	Jessian (H. G. Engine)	53-4 31-6	5-29 3-13	.29	.34	.05	17-3	The actual tests on this span included speeds up to 55-6 M. P. HR. at which speed the impact was 11-5 per cent. See Record Sheets Nos. 8, 9.
78-33	Budha Nullah (H. G. Engine)	53-8	5-33	.33	.41	.08	24-3	At 57 M. P. HR. (max. speed) the impact recorded was 18-6 per cent. See Record Sheets Nos. 1, 2.
78-33	Budha Nullah (M. Class Engine) (Modified).	32-9	2-57	.29	.33	.04	13-8	Highest test speed was 52-8 M. P. HR. at which speed the impact was 4-7 per cent. See Record Sheets Nos. 2, 4.
78-33	Budha Nullah (S. G. Engine)	4-1	3-76	.268	.30	.032	12-0	Highest test speed was 51-23 M. P. HR. at which speed the impacts in two tests were 6-3 per cent. and 3-0 per cent. respectively. See Record Sheets Nos. 2, 4, 6.

10816	East Beyne (H. G. Engine)	37	3.66	.428	.537	.109	25.4	Highest test speed was 49.6 M. P. H.R. at which speed the impact was 19 per cent. See Record Sheets Nos. 35, 36.
10816	East Beyne (H. G. Engine)	39.7	3.93	.432	.530	.098	22.6	Highest test speed was 56.7 M. P. H.R. at which speed the impact was 8.8 per cent. See Record Sheets Nos. 36, 38.
136	East Beyne (H. G. Engine)	38.6	3.82	.407	.52	.113	27.8 (1st Series)	Highest test speed was 52.4 M. P. H.R. at which speed the impact was 3.2 per cent. See Record Sheets Nos. 26, 27, 30.
		38.6	3.82	.403	.5	.097	24.0 (2nd Series)	
206	Sutlej (H. G. Engine)	44.9	4.45	.367	.44	.073	20.0	See Record Sheets Nos. 15, 16, 18.
		31.1	3.08	.42	.51	.09	21.5	
		20.4	2.02	.712	.82	.108	15.2	See Record Sheets Nos. 57, 58.
		20.4	2.02	.70	.795	.095	13.6	See Record Sheets Nos. 61, 58.
	Adamwahan (H. G. Engine)	21.3	2.11	.702	.88	.178	25.3 (Marked vibrations.)	See Record Sheets Nos. 64, 68.
257		23.1	2.29	.725	.795	.070	9.6	See Record Sheets Nos. 63, 64.
	Adamwahan (H. P. Engine)	30.8	2.35	.66	.725	.065	10.0	See Record Sheets Nos. 82, 84.
		28.8	2.2	.66	.73	.07	10.6	See Record Sheets Nos. 83, 84.
		21.3	2.11	.532	.60	.068	12.8	See Record Sheets Nos. 108, 109.
		39.7	3.93	.52	.599	.079	15.2	See Record Sheets Nos. 85, 86.
	Kotri (H. G. Engine)	21.3	2.1	.522	.61	.088	16.9	See Record Sheets Nos. 89, 90.
358		22.0	2.18	.545	.66	.115	21.0 (Marked vibrations.)	See Record Sheets Nos. 94, 95.
		22.3	2.21	.538	.67	.132	24.5 (Marked vibrations.)	See Record Sheets Nos. 99, 100.
	Kotri (H. P. Engine)	27.2	2.08	.475	.52	.045	9.5	See Record Sheets Nos. 104, 105.
	Kotri (S.G. Engine)	42.6	3.91	.402	.46	.058	14.5 (Distinct vibrations.)	See Record Sheets Nos. 106, 107.

Critical Frequencies—Calculated and Observed.

Span in feet.	Test Engine Type.	Maximum impact % recorded in tests.	LOADING PER FT. RUN IN TONS.			Static deflection in inches. d.	Period of vibration under test load $T = \sqrt{\frac{w+p}{P} \times \frac{d}{12}}$	CRITICAL FREQUENCY. no.		CRITICAL SPEED M. P. HR.	
			Dead Load of Span. w.	Equivalent Load of Engine. P.	Standard live load for which bridge is designed. P. max.			Calculated $= \frac{1}{T}$.	Observed.	Calculated ft. $\times C * \times .682$.	Observed.
1	2	3	4	5	6	7	8	9	10	11	12
16-16 33-33 41-33 56-66 78-33 78-33	H. G. H. G. H. G. H. G. H. G. M. (Modified.)	10-5 31-0 9-8 17-3 24-3 13-8 (At 32-9 M. P. HR.)	.21 .337 .372 .435 1-16 1-16	3-0 2-6 2-43 2-20 2-07 1-565	4-09 3-10 2-89 2-65 2-47 2-47	.095 .210 .246 .29 .33 .29	.092 .141 .154 .170 .207 .205	10-9 7-1 6-5 5-9 4-83 4-88	...	110 72 65-6 59-6 48-75 63-6	Not reached. Not reached. Not reached. Not reached. Not reached. (Max. reached =52-8 M. P. HR.) Not reached. (Max. reached =51-2 M. P. HR.)
78-33	S. G.	12-0 (At 41 M. P. HR.)	1-16	1-543	2-47	.268	.198	5-05	...	55	
108-16 108-16 136 206 257 257 358 358 358	H. G. H. G. H. G. H. G. H. G. H. P. H. G. H. P. S. G.	25-4 22-6 27-8 24-0 21-5 25-3 10-6 24-5 9-5 14-5	1-21 1-21 1-32 1-32 1-64 1-76 2-62 2-62 2-62	1-66 1-66 1-40 1-40 .98 .825 .74 .597 .534 .427	2-32 2-32 2-21 2-21 2-01 1-52 1-52 1-41 1-41 1-41	.428 .432 .407 .403 .42 .702 .66 .538 .491 .483 .402	.248 .249 .257 .256 .306 .428 .431 .491 .483 .489	4-03 4-01 3-9 3-91 3-27 2-33 2-32 2-04 2-07 2-05	3-66 3-93 3-82 3-82 3-08 2-11 2-2 2-21 2-08 3-91	40-7 40-48 39-4 39-5 38-6 31-1 21-3 28-8 22-3 27-2 42-6	

* C = Circumference of driver in feet.

SHEET No. 3.

I. R. B. C.—IMPACT TESTS, 1921.

Comparison of Recorded and Calculated Impact.

Span in feet.	Maximum Recorded Impact per cent.	Weight of overbalance at crank pin centre. (lbs.) M	Radius of Crank (inches). r	Hammer Blow at one Rev. per Sec. (tons). P_1	Total load per ft. run (tons). ($w + p$)	Static Deflection in inches. d	Circumference of Loco Drivers (feet). C	Calculated Impact per cent. $I = \frac{1050 \times P_1}{(w+p) \times C \times d}$
(1)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
16.16	10.5	715	13	.604	3.21	.095	14.8	...
33.33	31.0	715	13	.604	2.937	.210	14.8	69
41.33	9.8	715	13	.604	2.802	.246	14.8	62
56.66	17.3	715	13	.604	2.635	.29	14.8	56
78.33	24.3	715	13	.604	3.23	.33	14.8	40
108.16	25.4	715	13	.604	2.87	.428	14.8	35
136	27.8	715	13	.604	2.72	.407	14.8	39
206	21.5	715	13	.604	2.62	.42	14.8	39
257	25.3	715	13	.604	2.585	.702	14.8	24
358	24.5	715	13	.604	3.217	.538	14.8	25

Comparison of Recorded and Calculated Impact.

Span in feet.	Test No.	Maximum Recorded Impact. %	Weight of overbalance at crank pin centre (lbs.) M	Radius of crank (inches). r	Hammer Blow at one Rev. per Sec. (tons). P_1	Total load per ft. run (tons). $(w+p)$	Static Deflection in inches. d	Circumference of loco Drivers (feet). C	Calculated impact % $I = \frac{1050 \times P_1}{(w+p) \times C \times d}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
37 (B. R. No. 41)	1505	83	537 (7027)	13	452 f	6.24	.06	20.1	63
37 (B. R. No. 41)	1407	67	632 (7204)	14	573	7.35	.075	14.5	75.3
49 (B. R. No. 43)	1480	65	632 (7204)	14	573	6.4	.115	14.5	57.3
50	1010	67	856 (1746)	15	832	2.93	.23	16.5	78.6
59.17 (B. R. No. 40)	1381	77	537 (7027)	13	452 f	3.71	.15	20.1	42.4
59.17 (B. R. No. 40)	1357	53	632 (7204)	14	573	4.76	.18	14.5	48.4
65.42	676	48	786 (263)	15	764	2.64	.34	18.1	49.4
70 (Plate)	685	50	786 (263)	15	764	3.98	.17	18.1	65.5
70 (Plate)	858	26	1205 (1115)	14	1.09	3.69	.40	18.1	42.8
80	432	42	782 (304)	13	.66	3.13	.285	17.8	43.6
80	663	37 (Critical speed not reached).	1632 (939)	14	1.48	3.43	.265	22.1	77.4

100 (Plate)	1041	45	964 (2200)	16	1.00	2.82	.53	16.5	42.6
104 (Lattice)	1477	48	632 (7204)	14	.573	3.87	.24	14.5	44.3
112	83	42	Particulars not given	3.17	.30	16.5	...
124	819	38	(20)	14	1.09	5.5	.28	18.1	41
124	975	36	(1115)	15	.83	2.35	.38	16.5	59
132	154	29	856 (1746)	14	.686	2.34	.67	18.1	25.3
132	233	34	756 (1919)	13	.585	2.17	.56	22.1	22.9
132	240	18	694 (2527)	13	.585	2.0	.61	22.1	22.8
149	P	32	694 (2523)	14	1.09	5.5	.34	18.1	33.8
159.75	619	18	1205 (1115)	15	.764	2.80	.45	18.1	35.2
159.75	646	23 (Critical speed not reached).	786 (263)	14	1.48	2.80	.40	22.1	63.2
176.5	454	34	1632 (939)	14	1.48	2.46	.72	22.1	39.8
206	1288	21	1632 (925)	13	.452	4.48	.34	20.1	15.5
210	P	24	537 (7027)	14	1.09	2.68	.79	18.1	30.0
228	570	13	1205 (1066)	13	.31	2.45	.97	18.9	7.0
228	541	8	366 (185)	14	.628	2.4	.94	16.3	18.0
300	890	22	692 (147)	14	1.09	2.96	.97	18.1	22
440	P	5	1205 (1066)	13	.585	4.55	1.75	22.1	3.5
			694 (2543)						

Comparison of Recorded and Calculated Impact.

Span in feet.	Test No.	Maximum Recorded Impact, %	Class of Engine.	Hammer blow at one rev. per sec. P_1	Total load per ft. run (tons). $(w+p)$.	Static Deflection in inches. d	Circumference of loco. Drivers (feet). C	Calculated Impact % $I = \frac{1060 \times P_1}{(w+p) \times C \times d}$
(1)	(2)	(3)	(4)	(6)	(7)	(8)	(9)	(10)
10	A6	78	H4	...	3.59	.09	14.5	...
10	RNTM	110	N	...	4.08	.006	14.8	...
32.5	778A	61	H4	.53	3.41	.186	14.5	61
37.67	791B	52	H4	.53	3.47	.19	14.5	58
46	818B	37	H4	.53	2.76	.27	14.5	52
60 Up.	892C	20	H4	.53	2.91	.44	14.5	30
60 Down.	...	26	H4	.53	2.97	.38	14.5	34
63	730A	10	H4	.53	2.81	.39	14.5	35
80	95A	24	H4	.53	3.06	.29	14.5	43
91.25 (Double track).	623A	28	H4	.53	2.89	.25	14.5	53
226.67 (Double track).	395B	36	H4	.53	2.28	.435	14.5	39
226.67 (Double track).	400A	36	H4	.53	2.28	.42	14.5	40

Comparison of Recorded and Calculated Impact for the better balanced Indian Engines. SHEET No. 6-

Span in feet.	Test No.	Maximum recorded Impact.	Engine Type.	Hammer blow at one Rev. per sec. P ₁ .	Total load per ft. run in tons. (w+p).	Static Deflection in inches. d.	Circumference of loco. Drivers in feet. C	Calculated Impact % $I = \frac{1050 \times P_1}{(w+p) \times C \times d}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
(a) I. R. B. C. Tests, 1921 (N. W. Railway).								
788 (Deck). 108 (Pony truss). 136 (Through). 358 (Through). 368 (Through). 257 (Through). 788 (Deck). 567 (Deck). 136 (Through). 108 (Pony truss).	195 772 780 1509 1498 1143 173 353 755 765A	12.0 22.8 15.4 14.4 9.5 10.38 12.0 12.0 11.11 9.35	S. G. S. G. S. G. S. G. H. P. H. P. M (Modified) M M M	.312 .312 .312 .312 .458 .458 3 (Probable value) 3 3 3	2.70 2.50 2.40 3.05 3.15 2.50 2.70 2.90 2.40 2.50	.268 .35 .285 .402 .475 .66 .28 .24 .32 .352	16.02 16.02 16.02 16.02 19.2 19.2 19.1 19.1 19.1 19.1	28 23.3 30 17 17 15 22 24 21.5 19
(b) I. R. B. C. Tests, 1920 (B. N. Railway).								
100 (Deck). 80 (Deck). 40 (Deck). 150 (Deck). (Deck).	277 38 695 592	7.4 16 20 20	H. M. H. M. H. M. H. M. (Two engines).	.338 .338 .338 .676	2.65 2.30 2.20 2.5	.535 .31 .25 .59	14.6 14.6 14.6 14.6	17 34 44 33

TYPICAL ENGINES ON INDIAN RAILWAYS.

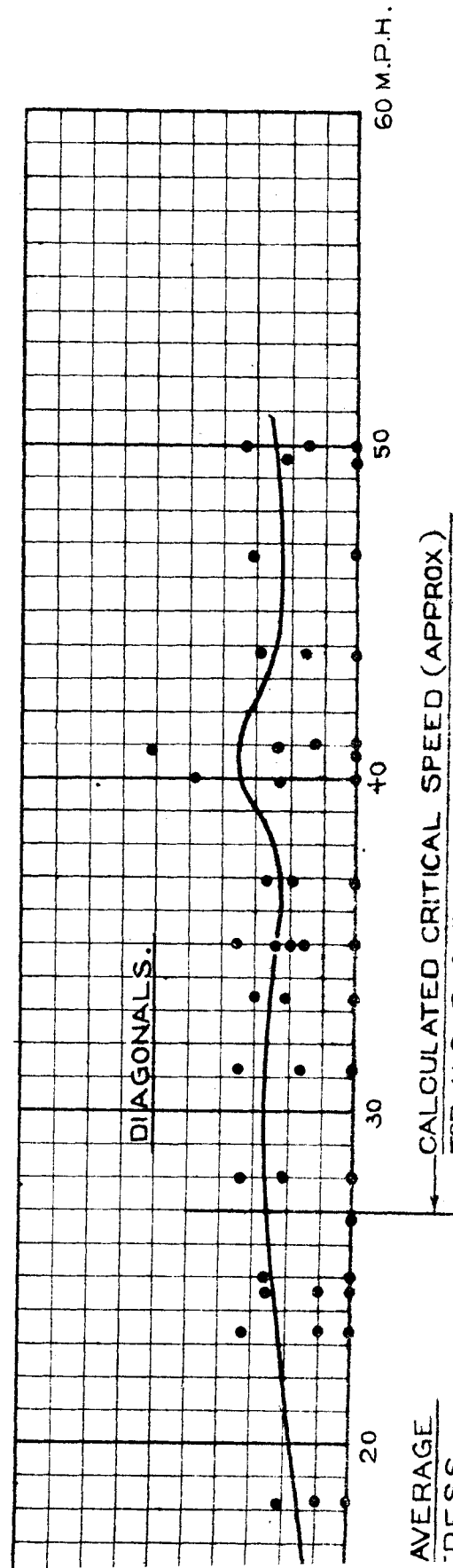
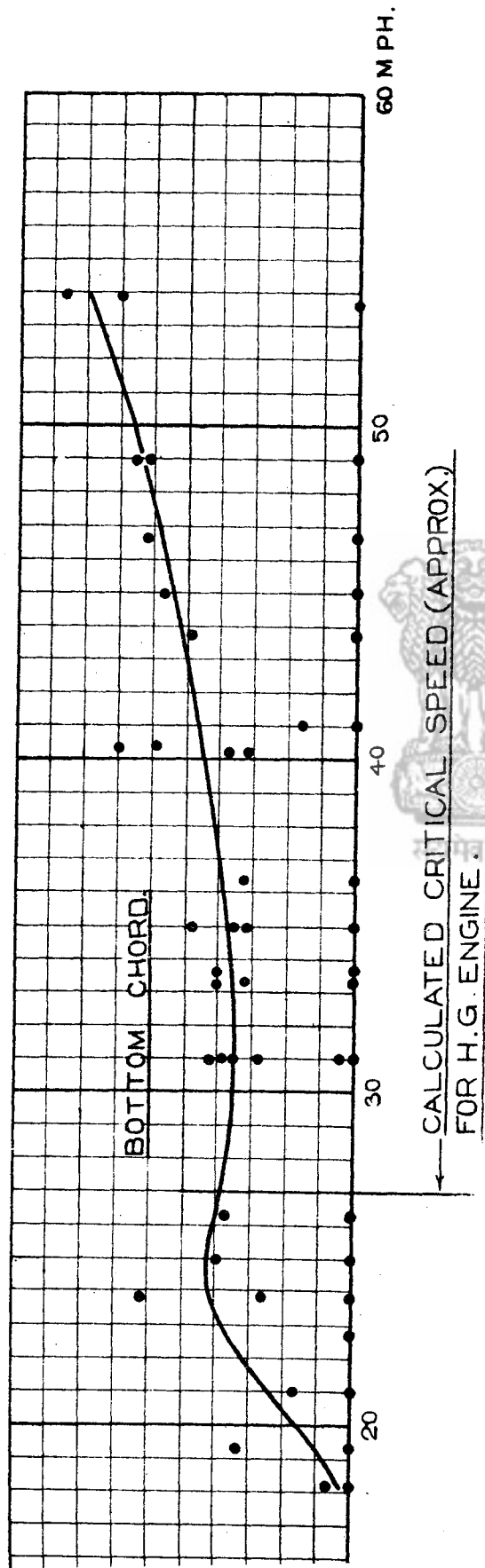
Hammer Blow = 000065 M x P.

SHEET No. 7.

Name of Railway Administration.	Type of Engine.	Axle load of drivers.	Weight per ft. run over Buffers.	RECIPROCATING WEIGHTS.						Total Hammer blow at one Rev. per sec. P ₁ .	Circumference of Loco. Driver.	REMARKS.
				Piston and Rod.	Cross head.	Small end of connecting rod.	Total reciprocating weight.	Weight of overbalance at crank pin. M.	Radius of Crank Pin. r			
		Tons.	Tons.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Inches.	Tons.	Feet.	
N. W. R.	Standard 2-8-0 (H. G.)	15.8	1.96	795	278	180 + 22 (Link pin.)	1073	715	13	604	14.8	
B. B. and C. I.	Non-standard 4-6-2 (P) Superheater.	19.75	2.21	458	300	204	960	640	14	583	19.35	
B. N. R.	2-8-0 (H. S.)	14.45	1.87	500	320	226	1024	462	13	39	14.7	
B. N. R.	2-8-0 (H. M.)	14.09	1.80	384	280	243	890	400	13	338	14.6	
O. and R.	4-6-0 (S. P.)	15.75	1.86	576	370	161	819	545	13	458	19.35	
O. and R.	0-6-0 (S. G. 2.)	16.7	1.67	396	557	370	13	312	16.1	
E. I. R.	2-8-2 (H. T.)	17.03	2.19	868	578	13	487	13.35	
E. I. R.	2-8-0	16.3	2.0	1004	668	13	562	14.8	
M. and S. M.	2-6-4	16.5	1.84	584	388	13	328	16.1	
G. I. P.	2-8-0	16.25	2.10	415	303	228	947	630	13	53	14.5	

TEST 1921.

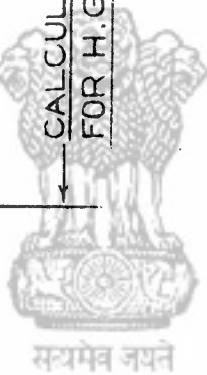
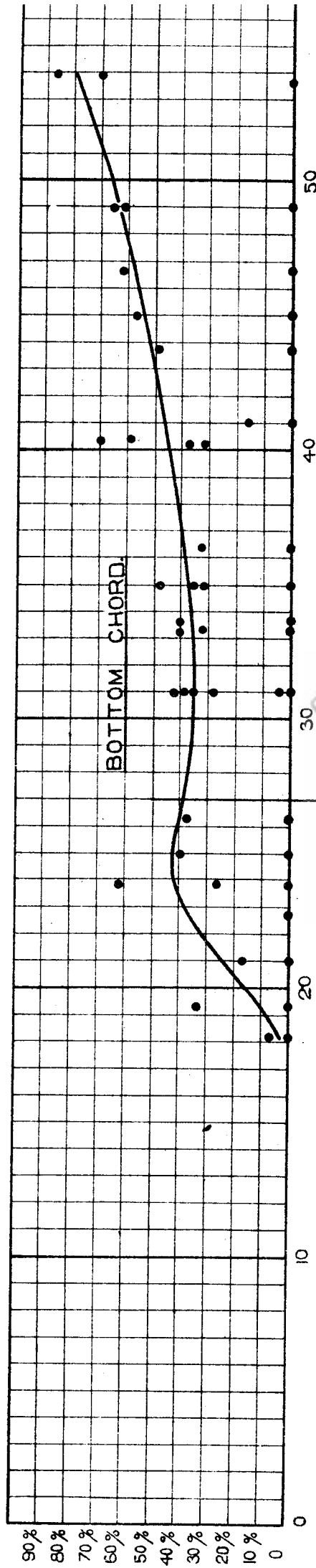
PLATE No. 1.



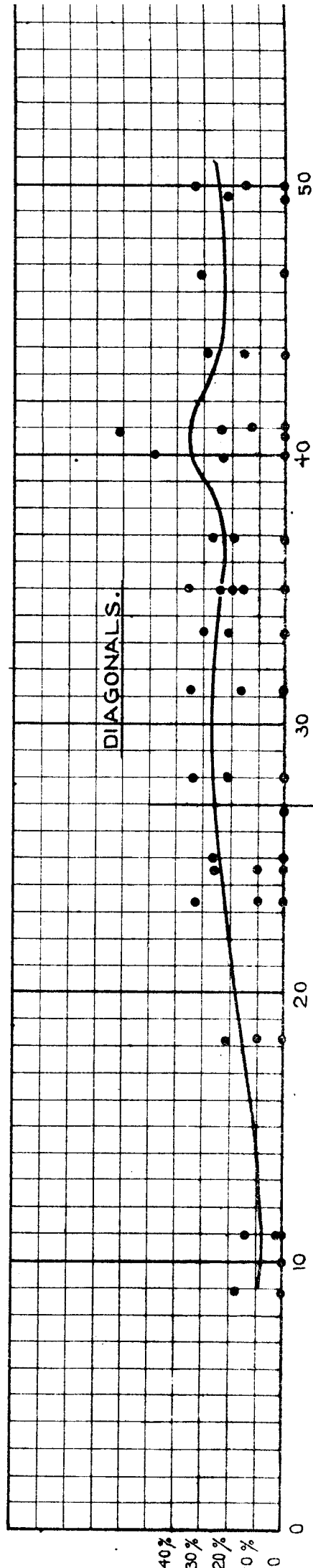
AVERAGE
OF 50

TEST 1921.

SUTLEJ BRIDGE
206 FEET SPAN.

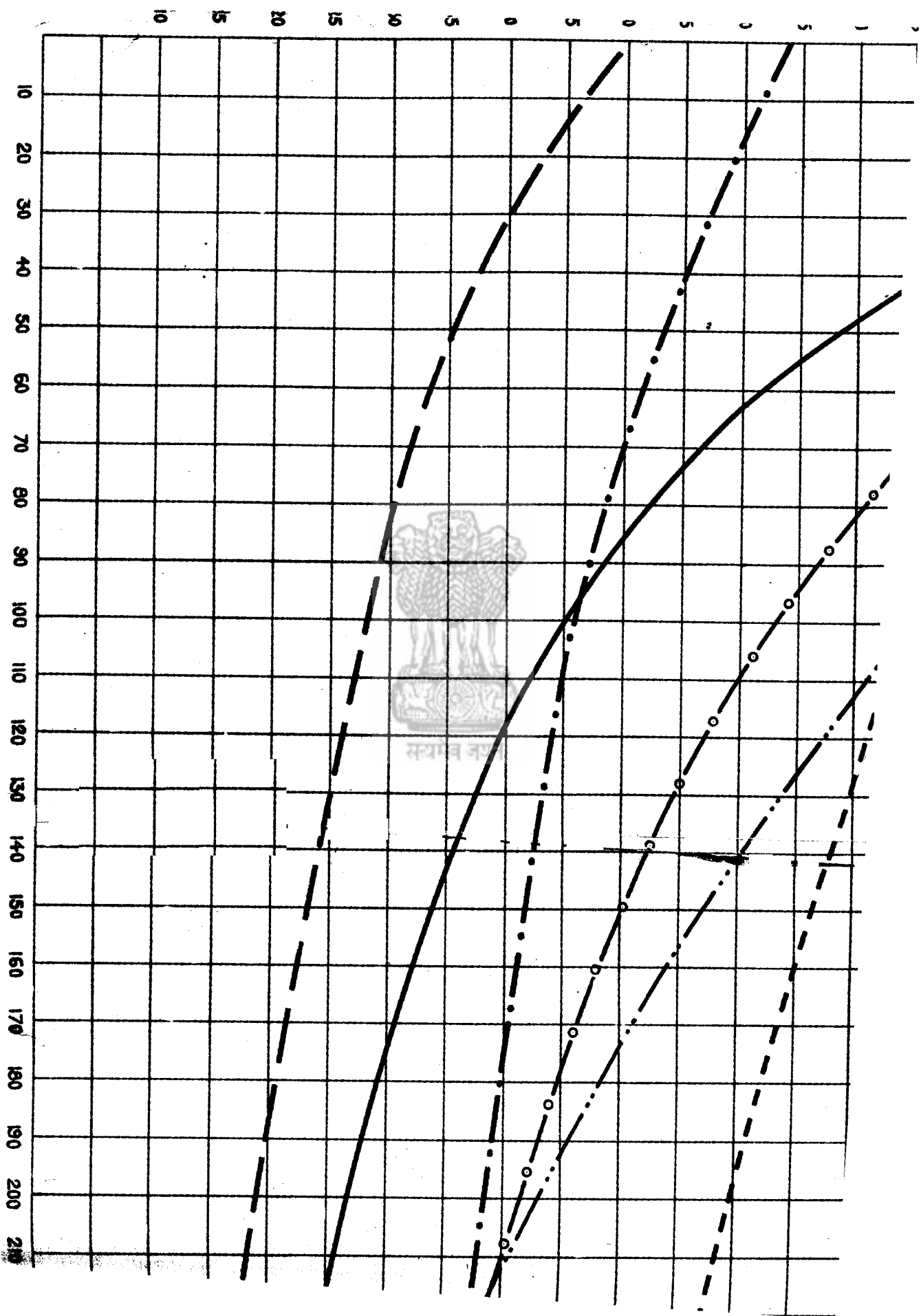


← CALCULATED CRITICAL SPEED (APPROX.)
FOR H.G. ENGINE.

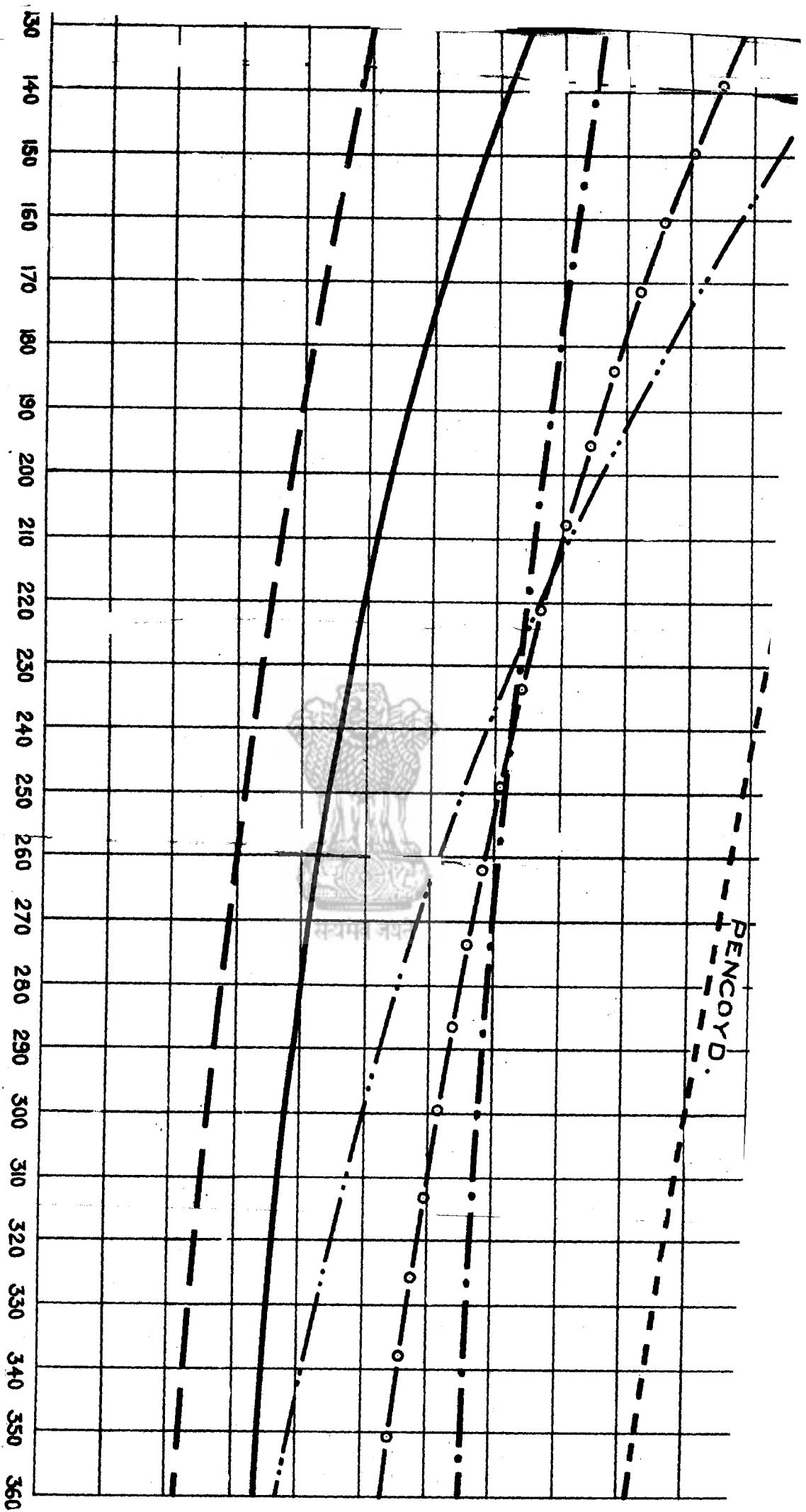


NOTE:- THE CURVES REPRESENT THE AVERAGE

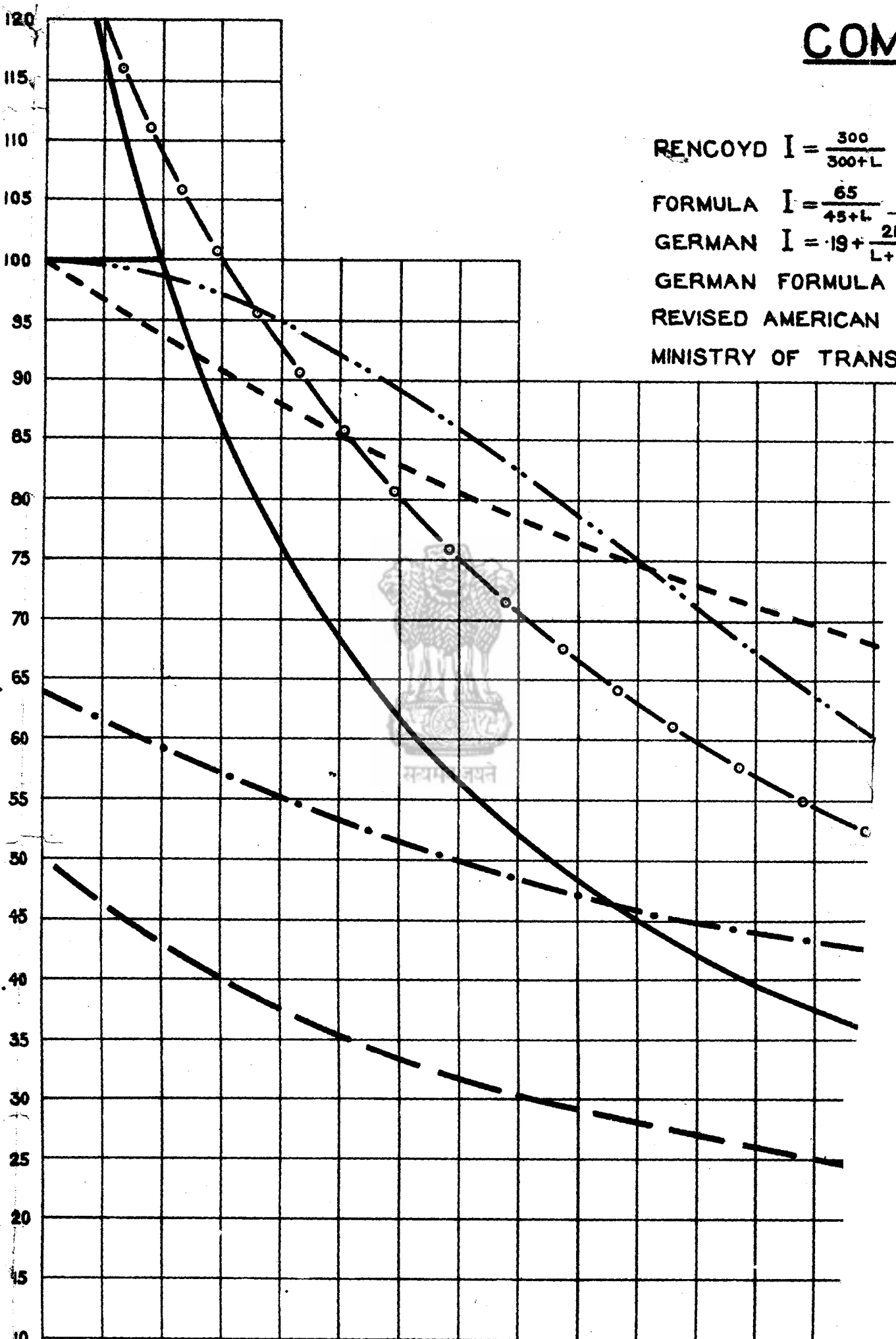
← CALCULATED CRITICAL SPEED (APPROX.)



SPAN IN FEET



COM



COMPARISON OF IMPACT

REFERENCES.

PENCOYD $I = \frac{300}{300+L}$ -----

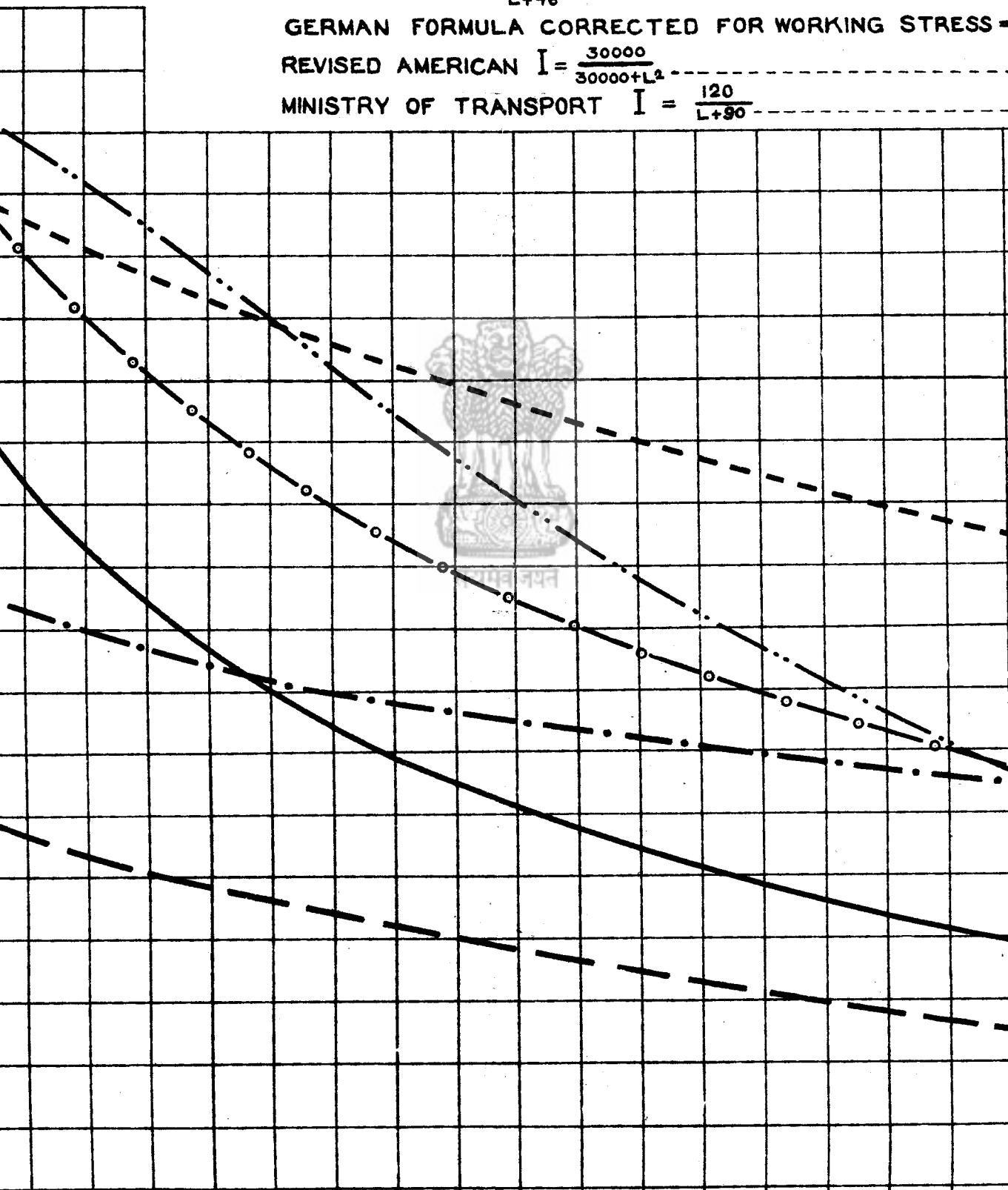
FORMULA $I = \frac{65}{45+L}$ -----

GERMAN $I = .19 + \frac{21}{L+46}$, WORKING STRESS 8.885 TONS SQ

GERMAN FORMULA CORRECTED FOR WORKING STRESS =

REVISED AMERICAN $I = \frac{30000}{30000+L^2}$ -----

MINISTRY OF TRANSPORT $I = \frac{120}{L+90}$ -----



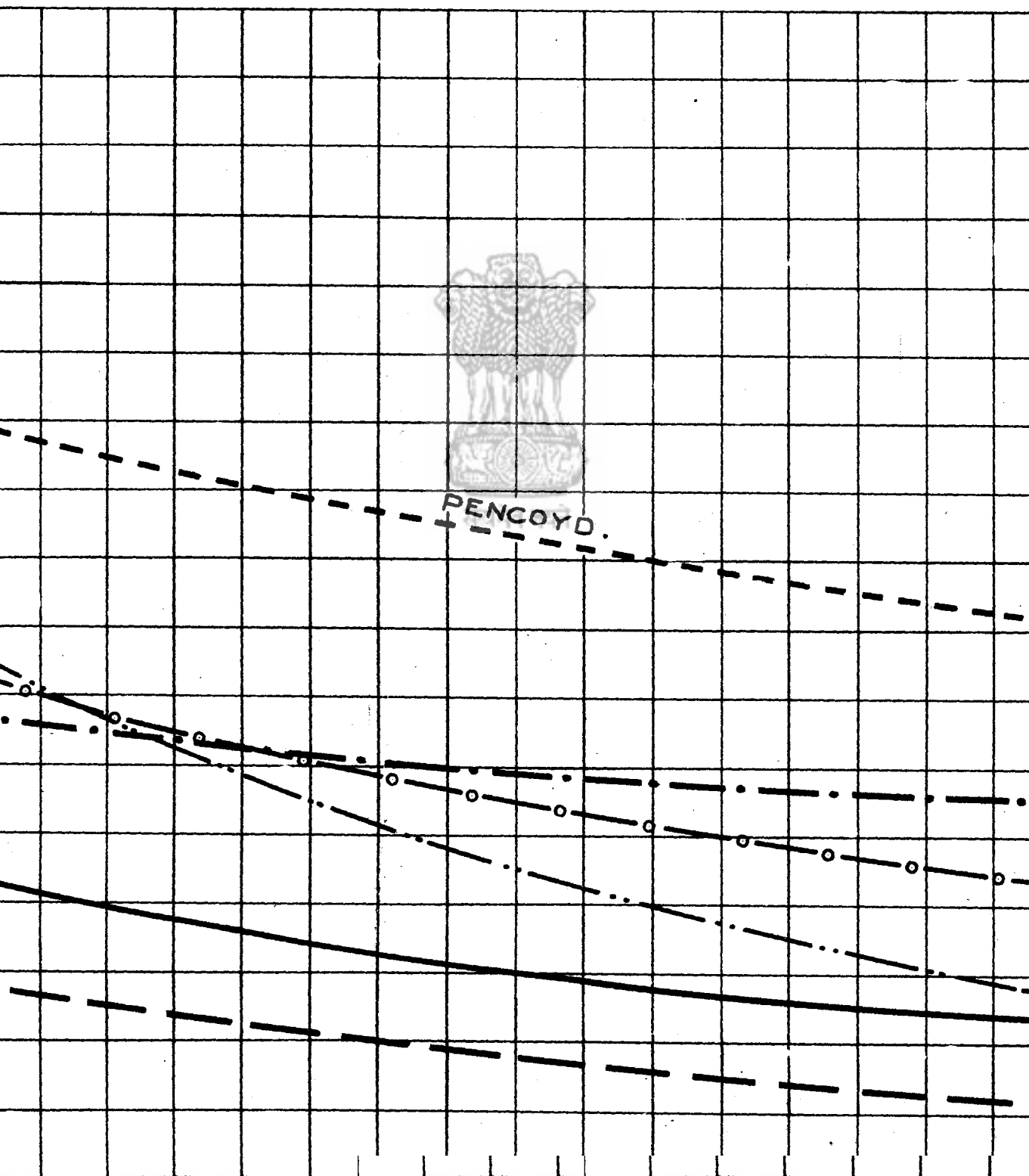
IMPACT CURVES.

SH

CES.

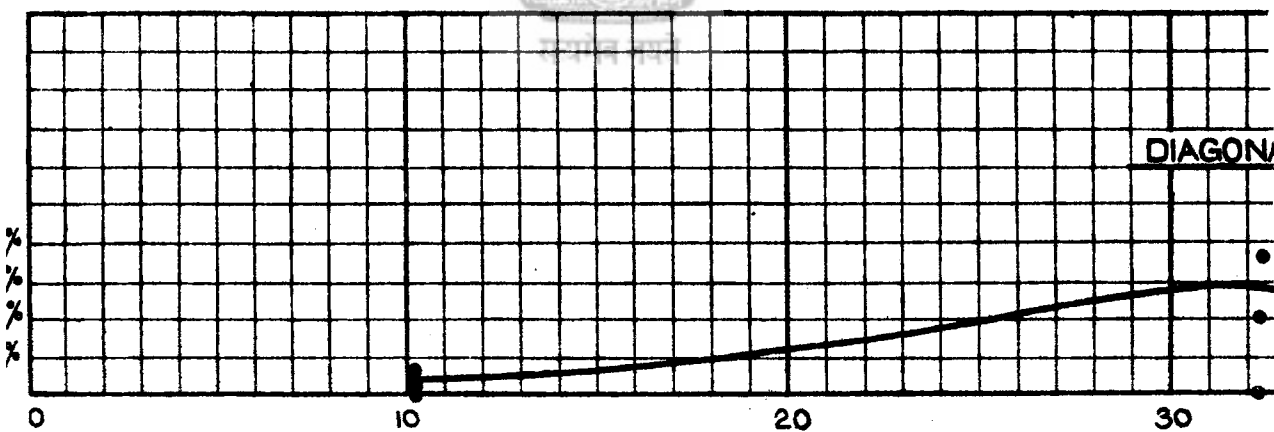
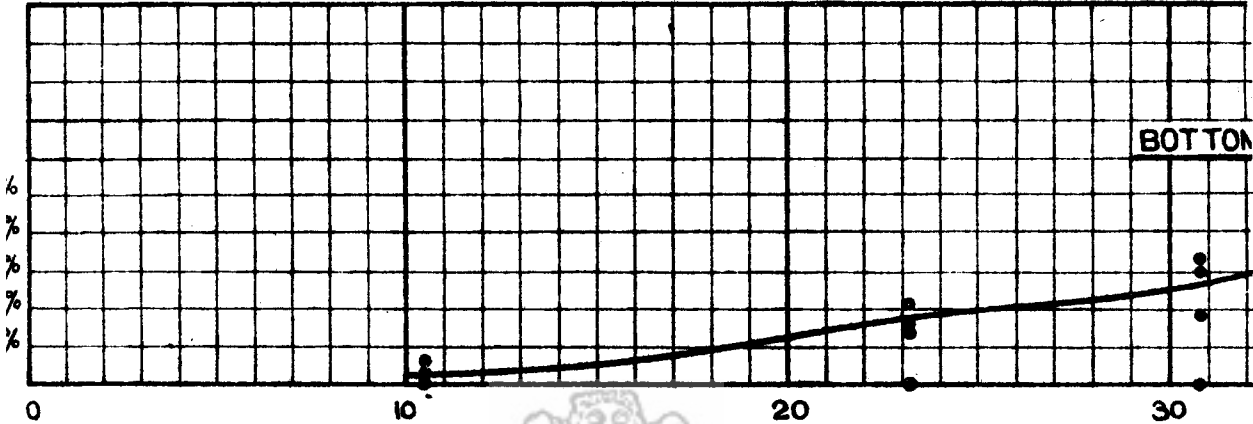
TONS SQ. INCH. -----
STRESS = 8 TONS SQ. INCH. -----

-----○-----○-----○-----



INDIAN RAILWAY BRIDGE TEST I

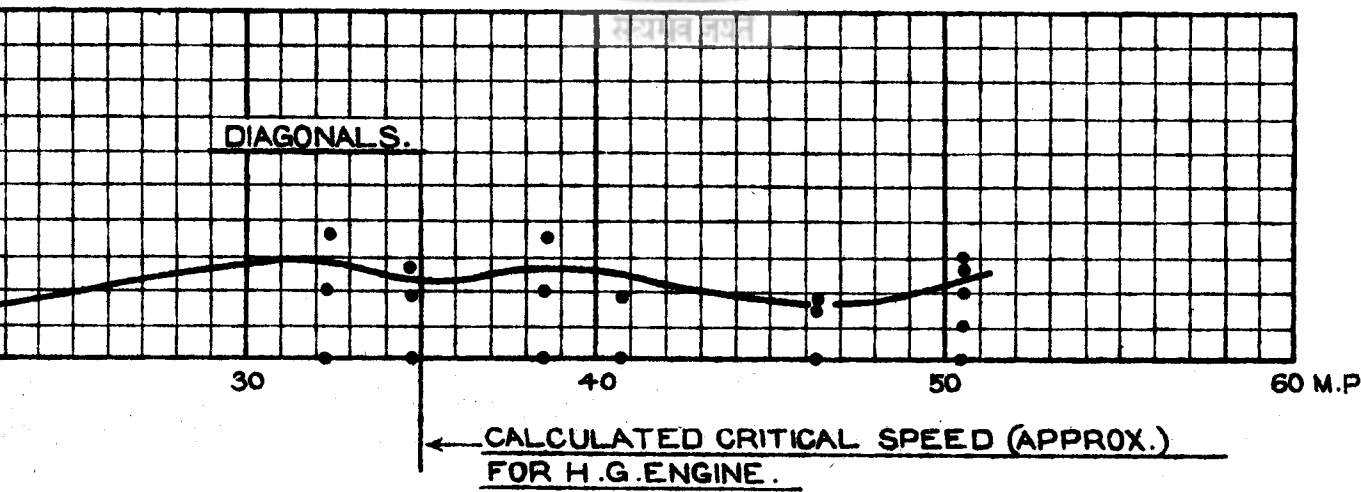
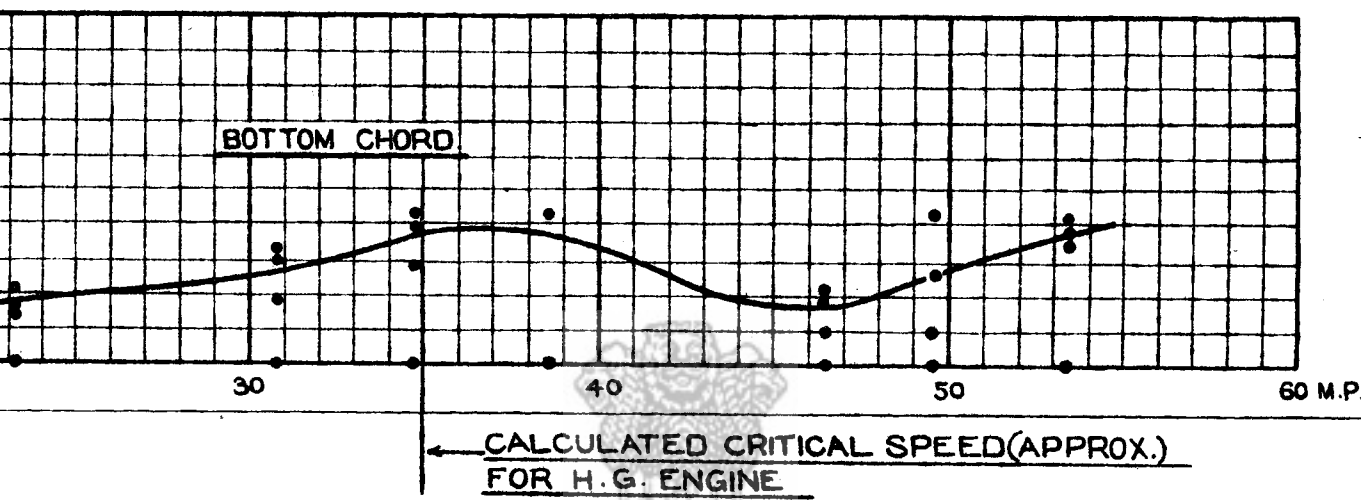
EAST BEYNE BRIDGE
136 FEET SPAN.



NOTE:- THE CURVES REPRESENT THE AVERAGE
PERCENTAGE INCREASE OF STRESS.

RAILWAY BRIDGE COMMITTEE
TEST 1921.

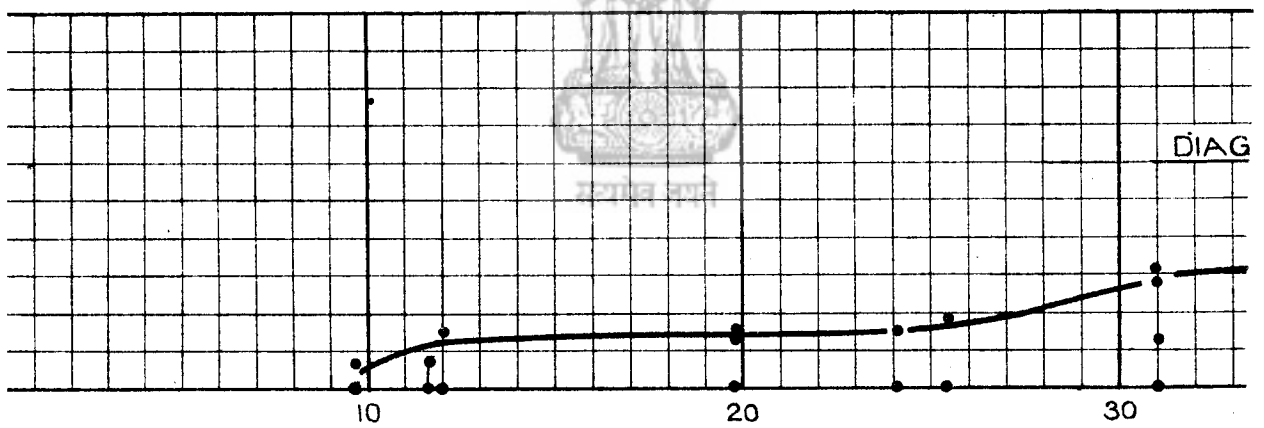
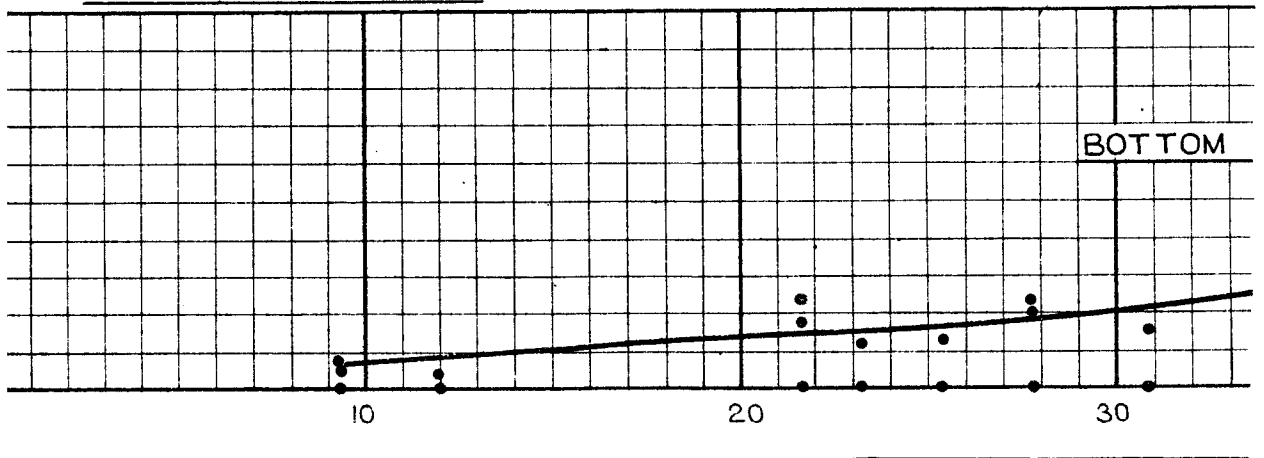
RENDEL PALMER & TRITTON
PLATE N^o 2.



INDIAN RAILWAY BRIDGE

EAST BEYNE BRIDGE
108.17 FEET SPAN.

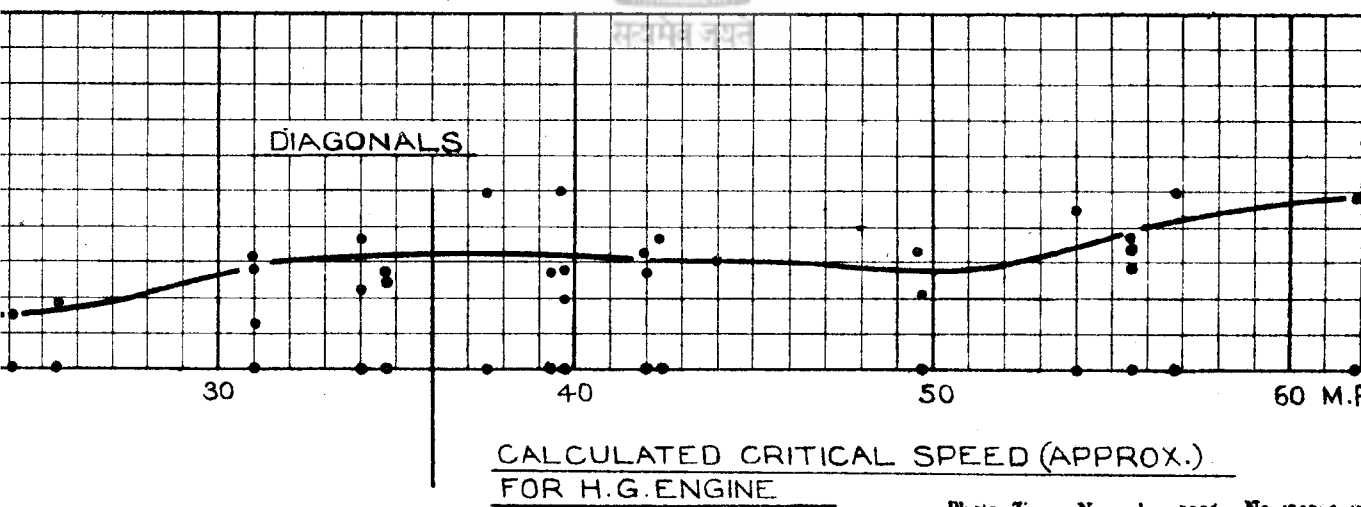
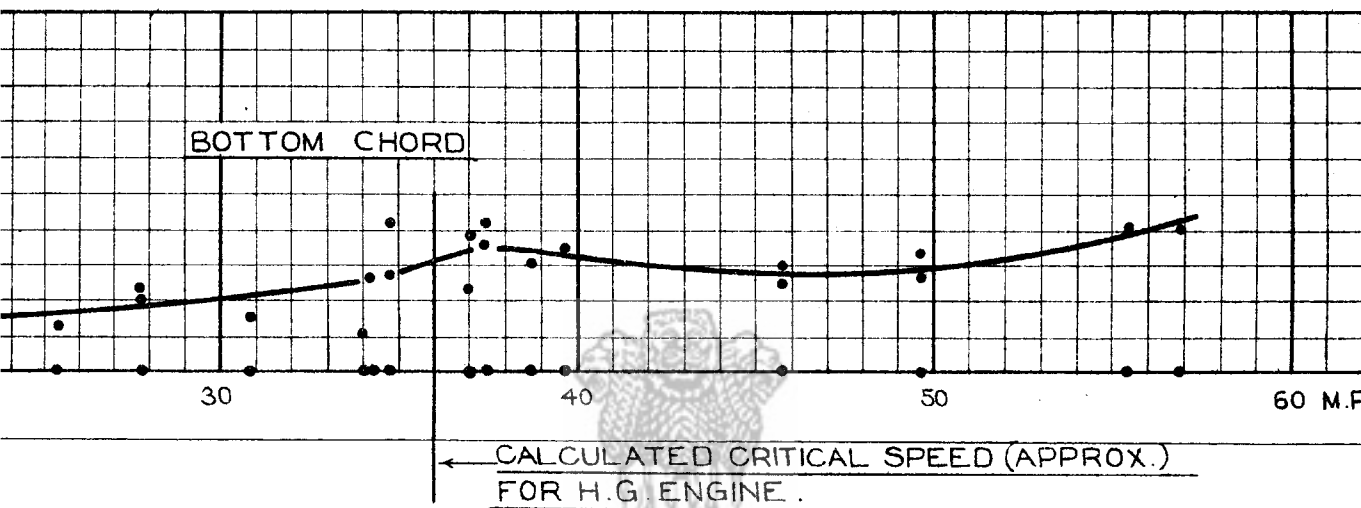
TEST 1921

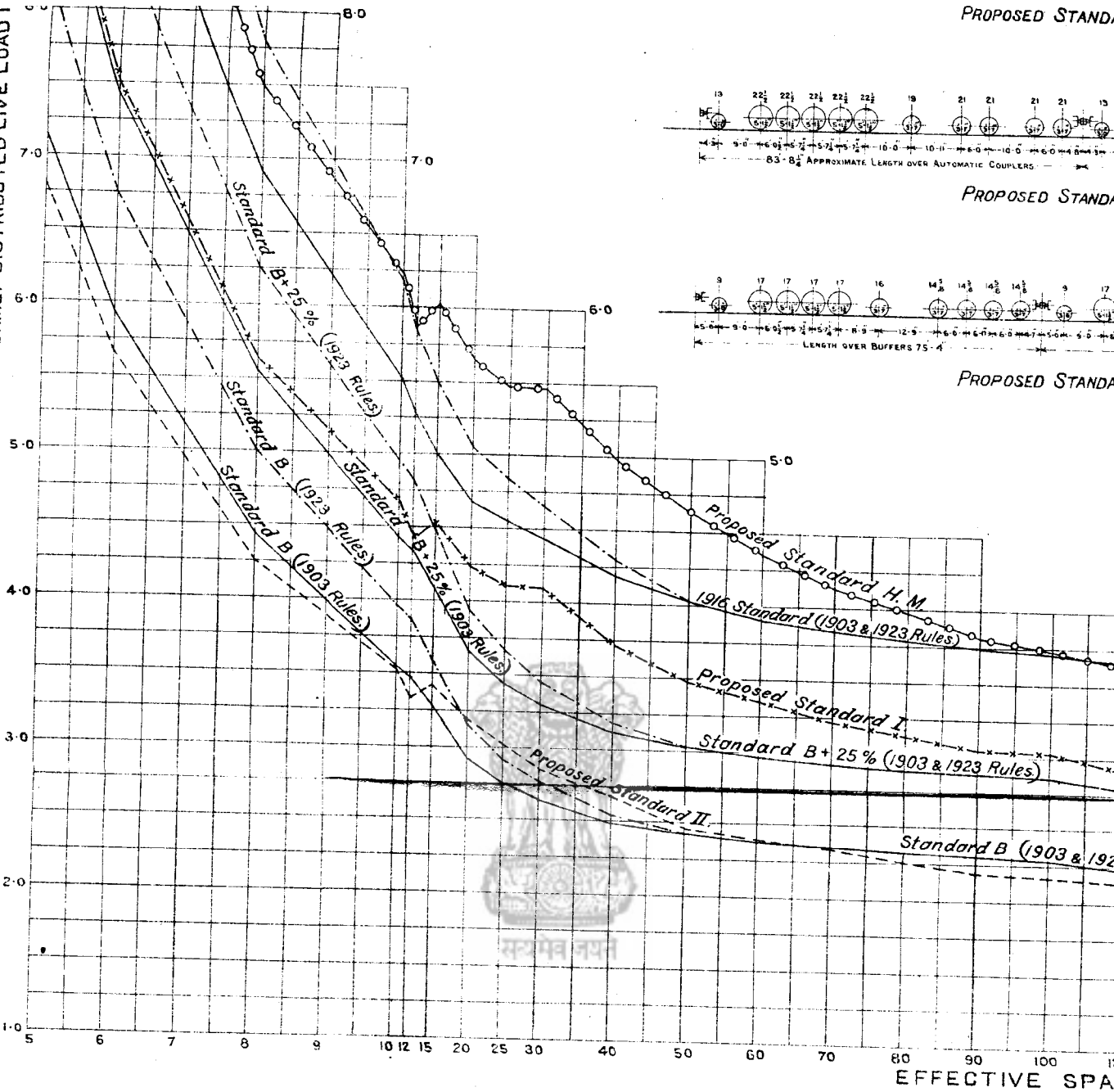


NOTE:- THE CURVES REPRESENT THE AVERAGE
PERCENTAGE INCREASE OF STRESS.

RAILWAY BRIDGE COMMITTEE
TEST 1921.

RENDEL PALMER & TRITTON
PLATE No. 3.





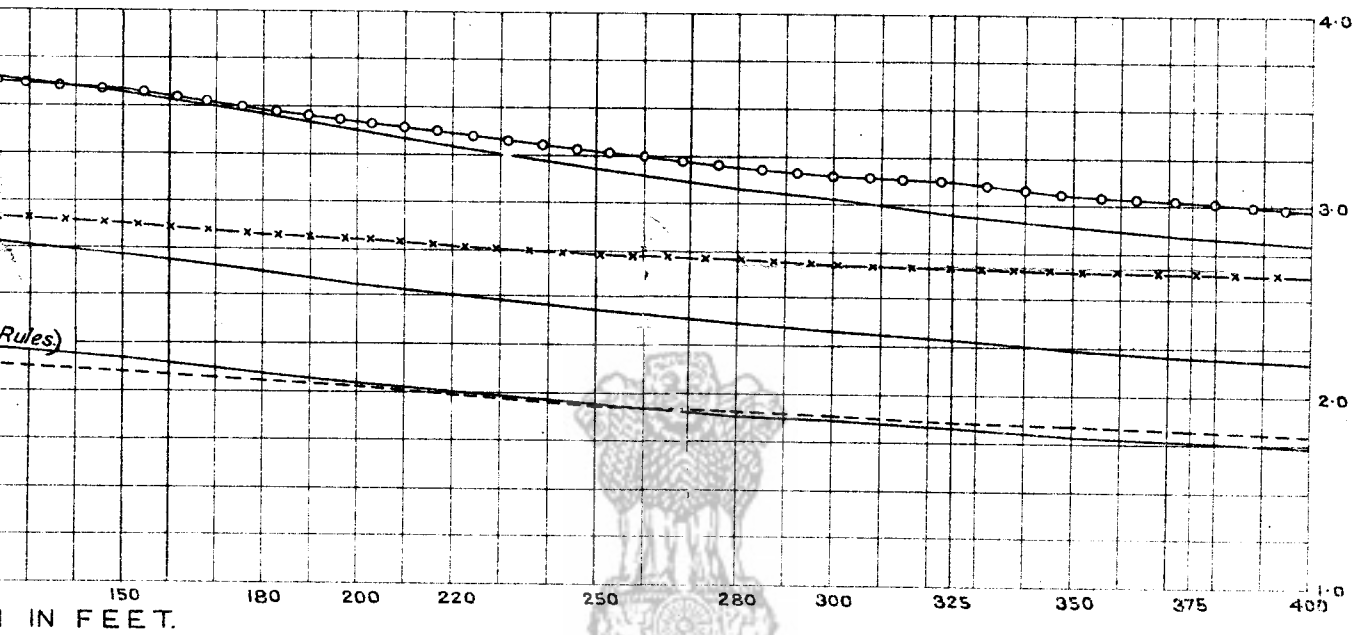
Note:—The horizontal scale has been made variable so as to ob

- 83'-8 1/2" APPROXIMATELY -

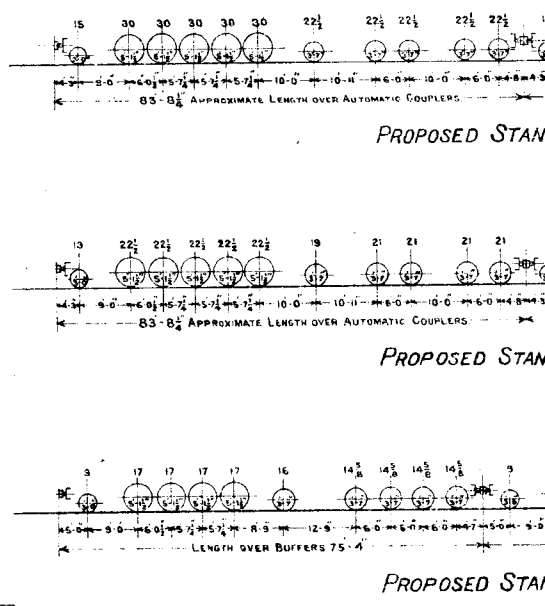
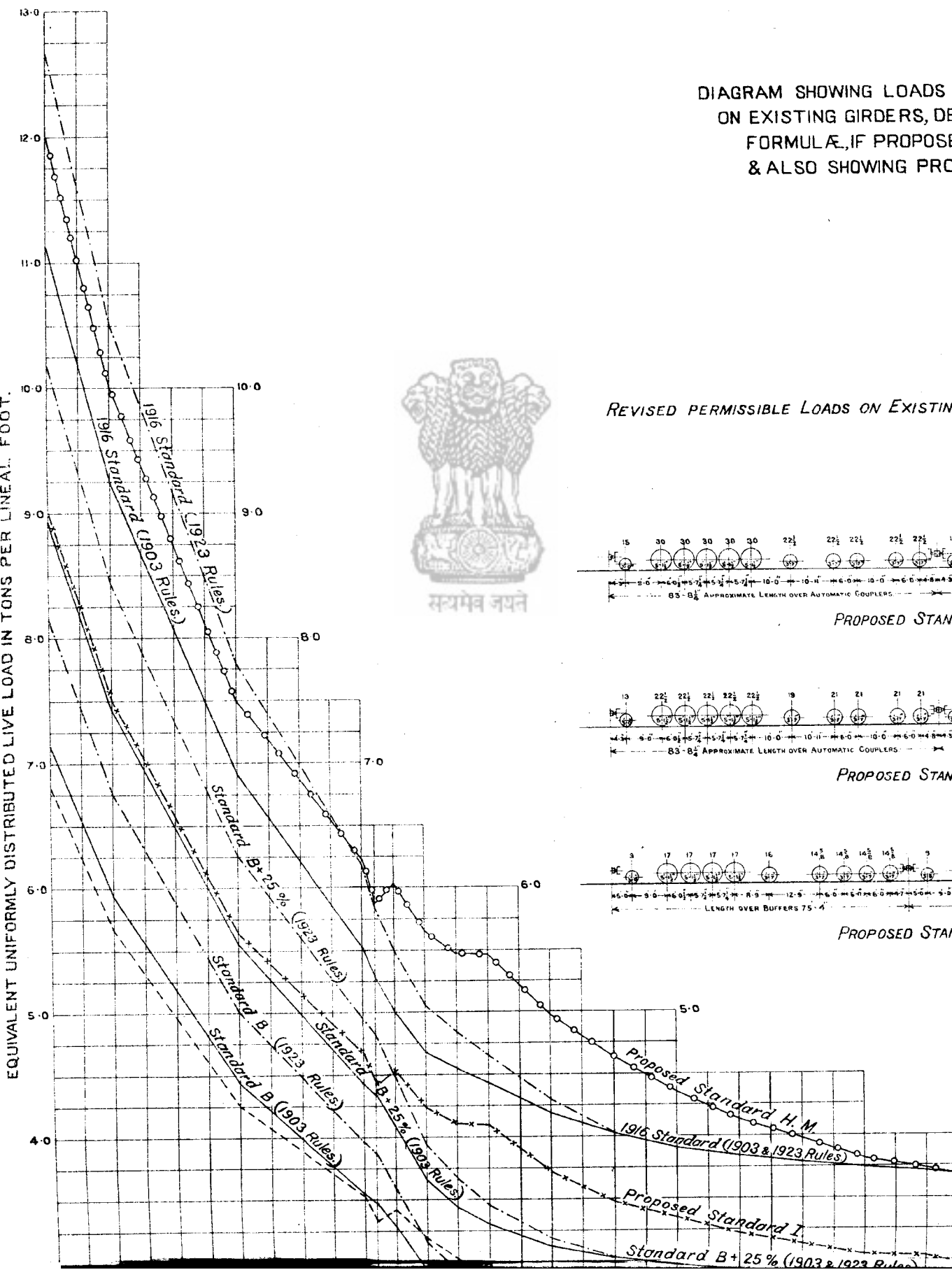
I. - TWO 2-10-2 TYPE GOODS ENGINES.



II. - TWO 2-8-2 TYPE (LIGHT) GOODS ENGINES.



main clearer curves, especially for spans of 5 ft. to 10 ft.

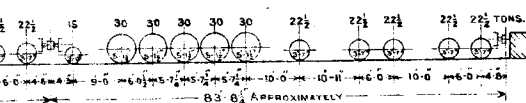


APPENDIX B. (2)

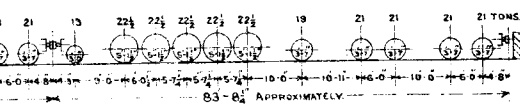
LOADS THAT MAY BE PERMITTED IN FUTURE
GIRDS, DESIGNED TO EXISTING & 1903 IMPACT
PROPOSED IMPACT FORMULA IS ADOPTED,
G PROPOSED STANDARDS OF LOADING.

EXISTING GIRDERS, IF PROPOSED IMPACT FORMULA IS ADOPTED.....

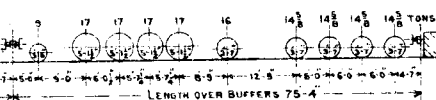
(1923 Rules)
(1903 Rules)



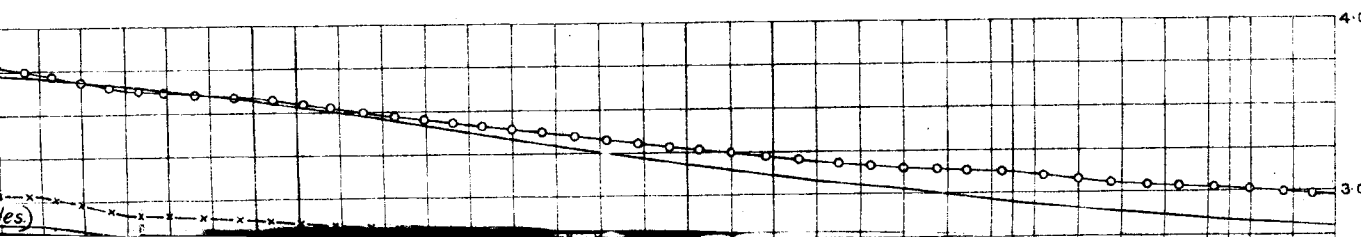
STANDARD H.M. — TWO 2-10-2 TYPE GOODS ENGINES.



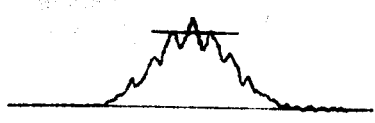
STANDARD I. — TWO 2-10-2 TYPE GOODS ENGINES.



STANDARD II. — TWO 2-8-2 TYPE (LIGHT) GOODS ENGINES.



N# 541.
DEAD SLOW.



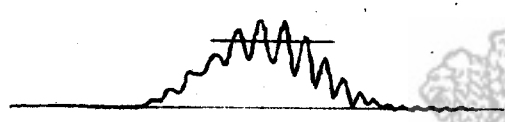
N# 577.
DEAD SLOW.

N# 930
DEAD SLOW.

TIME MARKING..
108.16 FT. SPAN.

I.R.B.C. - TEST N# 680.

N# 660.
DEAD SLOW.

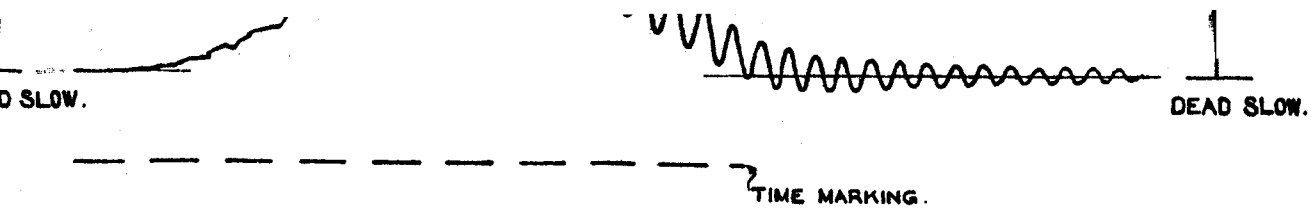


N# 696.
DEAD SLOW.

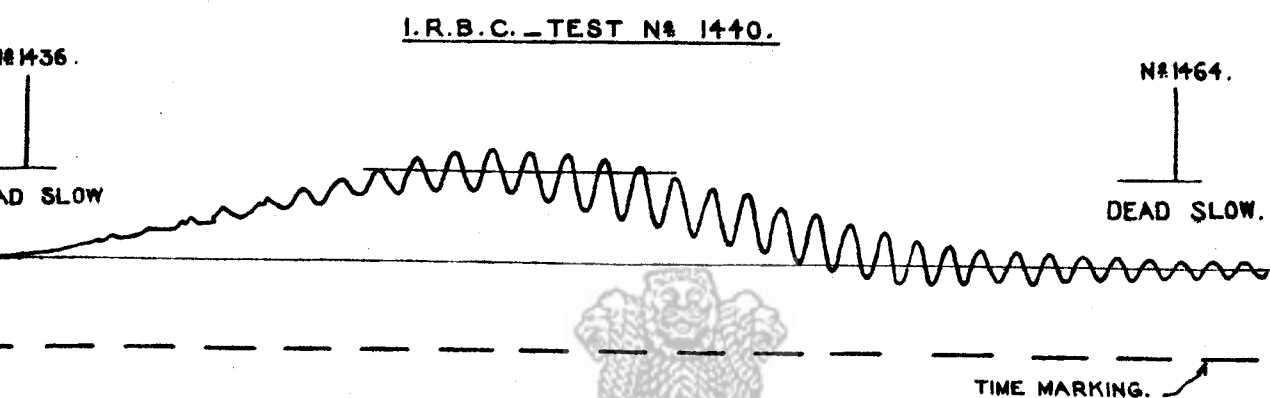
N# 1436.
DEAD SLOW.

TIME MARKING..
136 FT. SPAN.





257 FT. SPAN.



358 FT. SPAN.



TYPICAL DEFLECTOMETRIC AT CRITICAL S

I.R.B.C. - TEST N° 127.

N° 115.
DEAD SLOW.



N° 128.
DEAD SLOW.

N°
DEAD

--- TIME MARKING.
78.33 FT. SPAN.

I.R.B.C. - TEST N° 565.

N° 541.
DEAD SLOW.



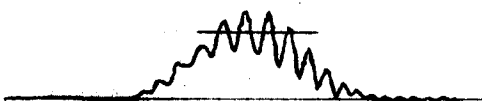
N° 577.
DEAD SLOW.

N° 930
DEAD SLOW.

--- TIME MARKING..
108.16 FT. SPAN.

I.R.B.C. - TEST N° 680.

N° 660.
DEAD SLOW.



N° 696.
DEAD SLOW.

N° 1436
DEAD SLOW

--- TIME MARKING.

ANEMOMETER DIAGRAMS
WIND SPEED.

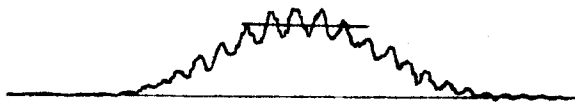
SHEET 8.

I.R.B.C. - TEST N° 433.

N° 421.



DEAD SLOW.



N° 453.



DEAD SLOW.



TIME MARKING.

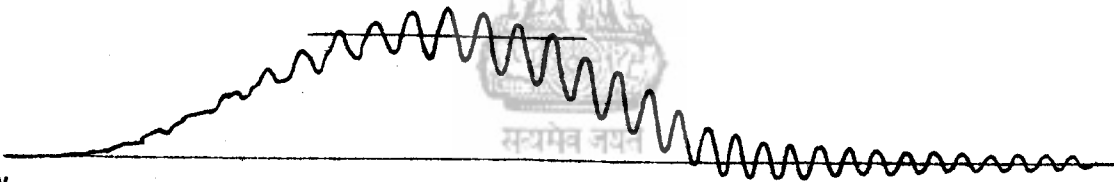
206 FT. SPAN.

I.R.B.C. - TEST N° 938.

N° 930.



DEAD SLOW.



N° 966.



DEAD SLOW.



TIME MARKING.

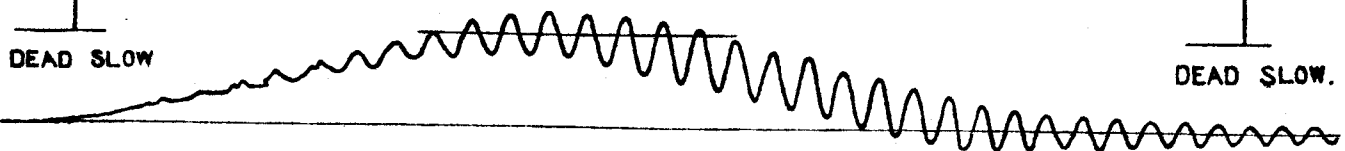
257 FT. SPAN.

I.R.B.C. - TEST N° 1440.

N° 1436.



DEAD SLOW



N° 1464.



DEAD SLOW.

